Township Water Supply

Zambia

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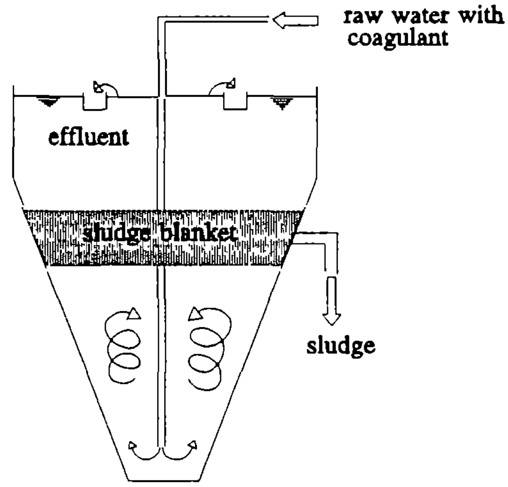
Upflow sedimentation tank



Delft University of Technology,

Centre for International Cooperation and Appropriate Technologie.

P. Holzhaus
N. Versteeg

January 1993

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SUMMARY

Within the long term technical cooperaton between the Department of Civil Engineering of the University of Zambia (UNZA) and the Delft University of Technology (DUT) a small scale project was defined together with the Ministry of Energy and Water Development, Department of Water Affairs.
In the period June to December 1992 an analysis and evaluation of township Water Supply in Zambia were made by two students of DUT.
Data were gathered within three months of fieldwork, in cooperation with an engineer from the Department of Water Affairs. The analysis of thirteen township water supply systems forms the basis of the evaluation.
The present situation is that especially the operation of water supply systems is very bad, which has several causes.
Operators do not have much insight in the working of a water supply system. Also the backstopping and supervision are not good.
Furthermore it is doubtfull if the nowdays used techniques for township water supply are operationally feasible at all.

TABLE OF CONTENTS

APPENDICES

1 INTRODUCTION

Modern water supply systems are available in some of the larger cities of Zambia. Taps in most of the houses deliver a regular flow of water to each consumer. In contrast in some remote places people still have to walk to the nearest stream to get water. Townships are the intermediate level between rural and large-scale urban living. This can also be said of the water supply systems in these townships. If not enough ground water is available, small water treatment plants are used to clarify surface water, and the distribution systems try to reach as many consumers as possible. Even so additional sources such as streams, lakes and shallow wells are still used for washing and cooking. In order to get an overview of the operation and state of these water supply systems and to find out what problems occur, two students of the Delft University of Technology, in cooperation with the University of Zambia and the Department of Water Affairs in Zambia, carried out a study, which included three months of field work, on township water supply. The results of this study are provided in this report.
The first chapter gives the framework and context in which the study was carried out. The second chapter gives an impression of the present general situation in Zambia. The basis for the main report is formed by appendices A and B, which give an inventory and an analysis of thirteen water supply systems. The results and conclusions together with additional information are reported in chapter three and four. Chapter three gives an overall view on the organization of township water supply and chapter four contains the results of the technical analysis. The last part of the report gives general conclusions and recommendations together with an evaluation of the study.

2 FRAMEWORK AND CONTEXT

2.1 General

A long-term technical cooperation project between the Department of Civil Engineering of the University of Zambia (UNZA) and the Delft University of Technology (DUT) has been running since 1980. The ultimate aim of the cooperation has been to reinforce the self-reliance of Zambia by contributing to the establishment of a fully Zambianised Department of Civil Engineering. The project supported course development and undergraduate teaching, including laboratory practical exercises, tutorials and final year projects. In combination with the above mentioned course developments the corresponding laboratories were equipped and made operational to provide adequate facilities for lab-exercises, practical exercises, final year projects and consultancy activities. During the 1985-1988 period a gradual shift was introduced towards environmental engineering. This was based on the growing recognition of the importance of safe water supply and sanitation for the development of Zambia combined with the fact that the Senate of the University of Zambia gave official approval for the development of a post-graduate program in this area within the Civil Engineering Department. The main objective of the third phase (1989-1992) of this project was the establishment of a fully operational environmental laboratory, capable of covering its teaching, research and consultancy duties. Coordination of the project is carried out by the Centre for International Cooperation and Appropriate Technology (CICAT) of the DUT.

2.2 Problem definition

Within the framework of technical cooperation between the UNZA and the DUT and together with the Ministry of Energy and Water Development, Department of Water Affairs a study was formulated, which consists of the analysis of township water supply systems in Zambia by students of DUT together with a Zambian engineer from Water Affairs. The period of study was from June to August 1992.
In township water supply systems problems occur with respect to:

* design
* operation
* maintenance & logistics
* financial management

To improve the water supply it is necessary to evaluate all aspects in order to approach the problem integrally.
At present the spare parts supply and financial management receive the most attention. Therefore the purpose of the study was to focus on design and operation aspects.

2.3 Objective

The objective of the project was to get an overview of the conditions of the surface water treatment and distribution systems used for township water supply in Zambia. This was done by means of an inventory and by the analysis of several systems which mainly included technical design and operational aspects.
The aim was to specify the problems in such a way that it would be possible to find the theoretical background and search for structural solutions.

In summary; the main aim was to describe and analyze the problems that occur in urban township water supply systems by means of:
* a field inventory
* finding causal relationships and theoretical explanations
* presenting the information by means of a report and a seminar.

2.4 Focus and limitations

At the start of the preparations for our field trip to Zambia we had to define the limits of the study.
It was decided that only surface water supply schemes would be visited because these schemes have an associated water treatment system. It was expected that this type of treatment system would imply extra difficulties with respect to design and operation.
The number of townships that could be visited was limited due to the time. In the selection of the kind of water supply systems we visited, the possibilities for reaching the township played an important role.
In the choosing which townships would be visited, we considered the interests of the Department of Water Affairs and the possibilities for reaching the township. Furthermore we intended to go to townships with all kinds of treatment systems and run by different institutions in order to get a varied view of the township water supply schemes in Zambia.

This report and its conclusions are limited to small scale (up to approx. 50,000 consumers) water supply systems that use surface water as the raw water source. Also because of limited time for field work, the conclusions are derived from the analysis of only thirteen townships but are considered as generally valid.
The study was done in the dry season, when surface water normally is less turbid and thus easier to treat than in the rainy season. The visit to each water supply system was for only about two days. For processes and data that cover a longer period it was not possible to take the measurements ourselves and therefore the only sources of information were reports by the operators and by the General Department of Water Affairs.
Another limitation was the pH and conductivity meter which was taken with us in the field. The metre was never accurate and the data could not be used.

2.5 Method

First a preliminary study was made in order to get familiar with problems that could be found in the field. A Checklist was made (see appendix A) in which a summary was given of all facts and details that were of interest concerning the schemes that were going to be visited.
Once in Zambia, contact was made with the University of Zambia and with the Department of Water Affairs in order to discuss our plans and the possibilities of transport and to collect more information on the townships that we intended to visit.

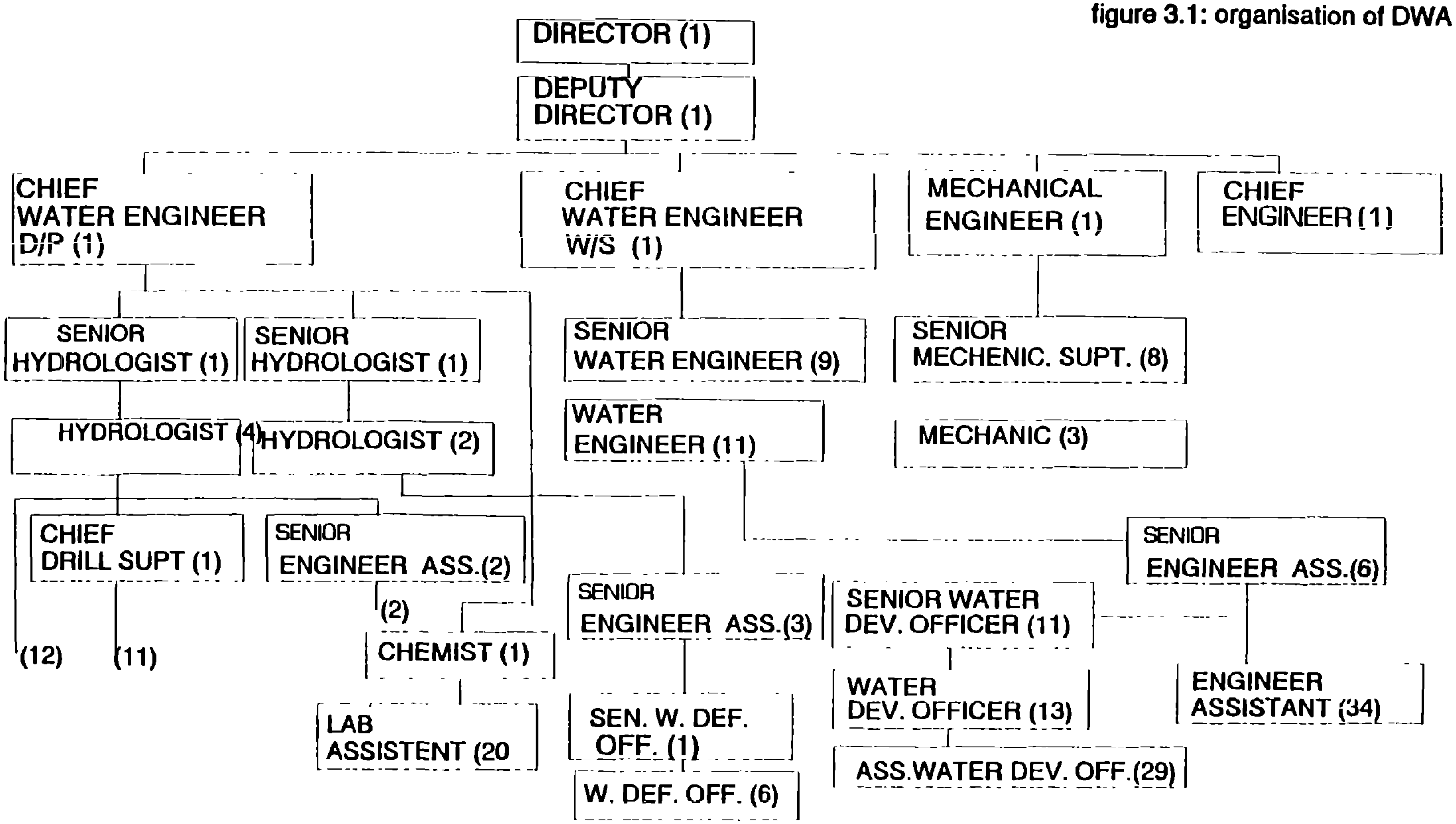
The fieldwork mainly consisted of collecting data by means of interviewing people and taking several measurements to get an overview of the management, operation and maintenance and technical aspects of the schemes. Two days were reserved to look through each system.
Thanks to the engineer and the driver of the Department of Water Affairs that joined us during the fieldwork, the people that were needed were quickly found and were very cooperative.

After the fieldwork the data were assembled and elabirated. All visited systems were analyzed and as far as possible tested against the theory of how they should work. For each system conclusions and recommendations were written.
These analyses per system and information about organisation and management were used to derive general conclusions.

PRESENT ORGANISATION CHART

DEPARTMENT OF WATER AFFAIRS JAN. 1991

figure 3.1: organisation of DWA



3 ORGANIZATION

3.1 Description

Framework

Nowadays the water supply schemes of townships are managed either by the Department of Water Affairs, by the District Council of the township or by a private company.
In general the District Councils are responsible for water supply both in urban and rural areas. Due to lack of qualified manpower, the Department of Water Affairs in the Ministry of Energy and Water Development is assisting the District Councils in different degrees in planning, implementation, operation and maintenance of water supply schemes in townships and in rural areas.
At present the Department of Water Affairs is responsible for all aspects of water resources conservation, planning, utilisation and advising the Government on all water related issues. In addition the Department handles 47 township water supplies which they took over from the District Councils in 1976. Furthermore the Department of Water Affairs is in charge of the construction of all rural water supplies.
The organisation of the water supply in 10 townships and all urban areas is still in the hands of the District Councils. They can get advice from the DWA when this is needed.

The Department of Water Affairs originates from the Department of Water Development and Irrigation which was established in 1947. The general responsibilities of the Department were the investigation and exploitation of the various sources of water. River flows and rainfall data were recorded in order to evaluate the potential supplies of water for domestic use, irrigation and power.
The name of the Department was changed in 1959 to its present name: the Department of Water Affairs.
After independence in 1964, the Government embarked on various development programs. Priority was given to the construction of new schools and hospitals, and to providing water supply facilities to the rural population. Due to implementation of these programs, the involvement of DWA was steadily increased.
Figures 3.1 and 3.2 show the present day structure of the Department of Water Affairs.

Operation and Maintenance

Preventive maintenance of the plants in the water supply schemes is, on the whole neglected. Only real defects are repaired if possible.
At most of the water supply schemes there is a shortage of spare parts and chemicals. Many chemicals are not available in the country and have to be imported from South Africa. Schemes that are run by the Department of Water Affairs have a central point in every province from where the spares and chemicals are distributed.

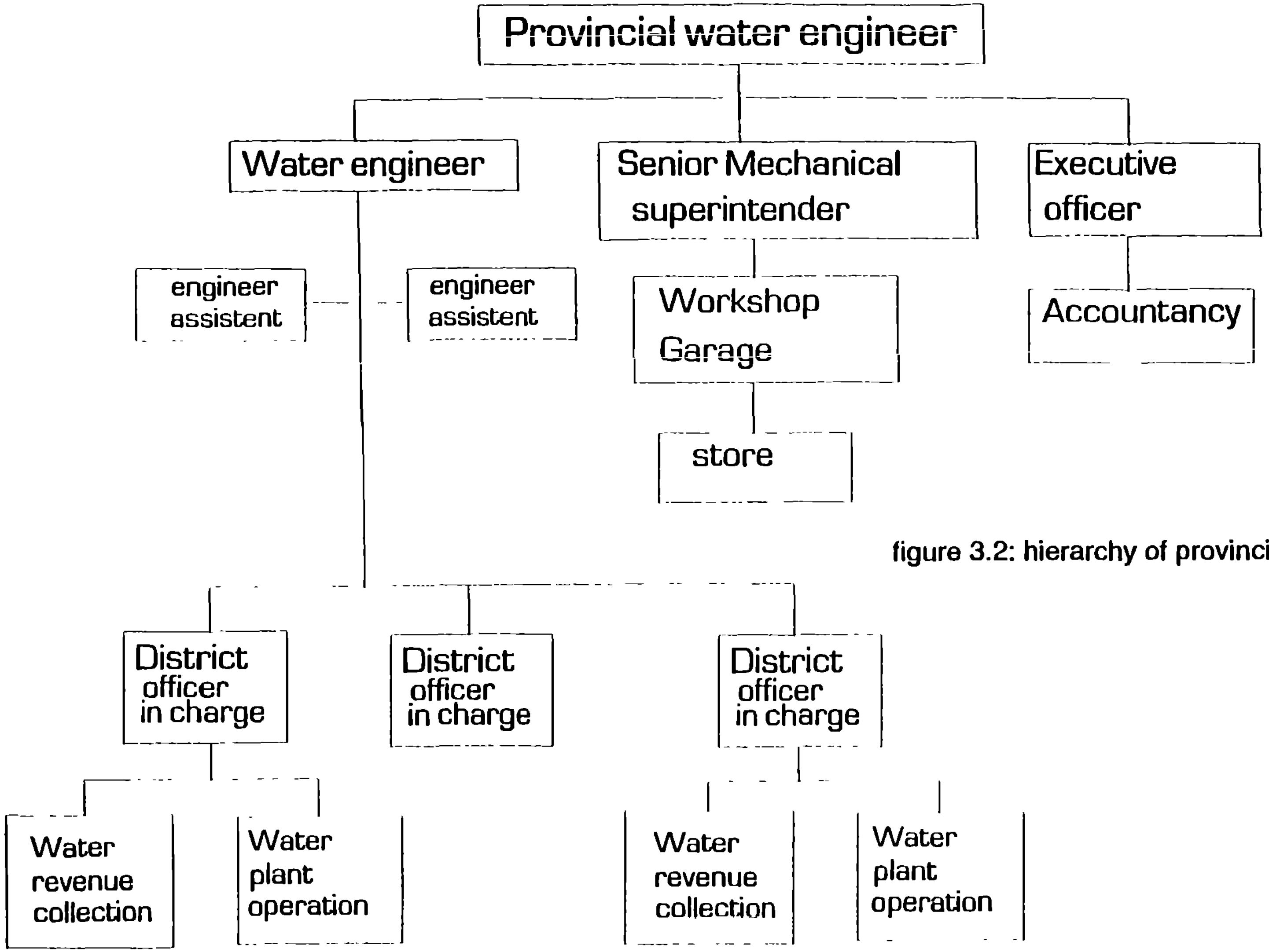
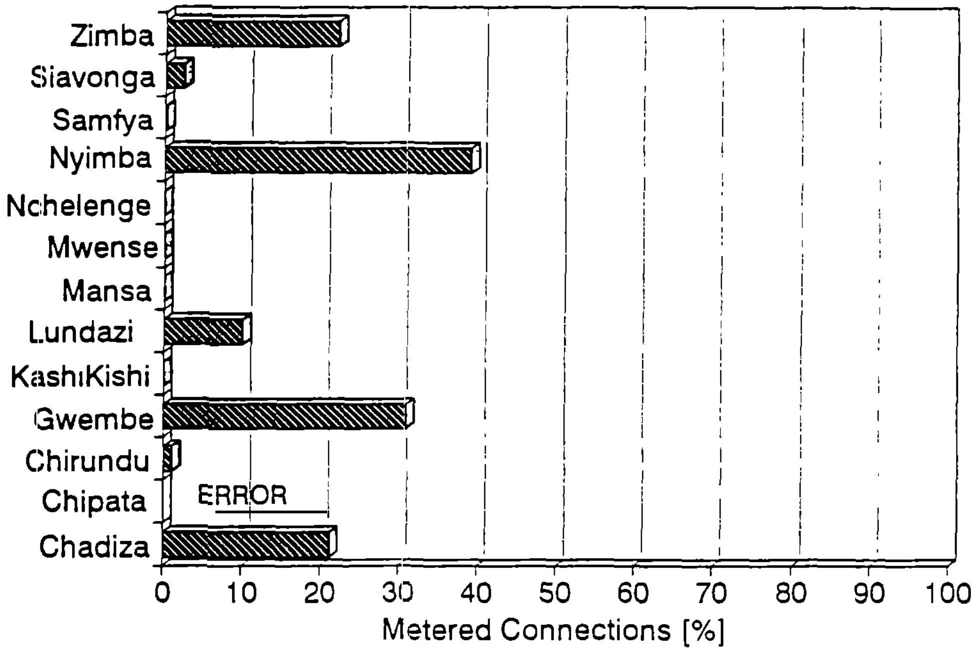


figure 3.2: hierarchy of provincial DWA

Schemes that are run by a District Council or a private organisation have some spares and chemicals in stock but mostly these items are ordered after they are needed.
Another problem concerning spare parts is that the pumps which are still used at the schemes are obsolete and a lot of them are not in production any more, so spares have to be specially made. This takes much more time.
There are only a few water meters at most water supply schemes. Most of the consumers pay a fixed amount of money every month independent from the amount of water they use. (fig.3.3)
In many townships the supply of water is irregular. There is not enough water produced to supply people for 24 hours a day so the time of supply is limited to certain hours.
There are rarely good maps of the area concerned. Both the size and situation of the mains such as the differences in heights are not known in most townships.

Management

There are different financial arrangements depending on the organisation in charge of the scheme.
Schemes that are run by the Department of Water Affairs are financially supported by the Ministry of Finances through the Headquarters in Lusaka and the Provincial Department of Water Affairs.
Revenue that is collected from the consumers of the produced water, goes back to the government via the Headquarters in Lusaka. This is not very stimulating because no matter how much money is collected, the township still financially depends on the Headquarters in Lusaka. This results in a remarkably low yield from the revenue collection (fig.3.4).
Since the beginning of 1992 the Department of Water Affairs has standard forms that have to be filled in monthly by the officers in charge and by the revenue collectors in order to get a better view on both the management and the financial aspects of each water supply scheme (See appendix E).
When the new forms are filled in correctly, the Headquarters in Lusaka will be able to justify to the ministry of Finances how much money they need to operate the schemes.
In the coming years a group of people within the Ministry of Local Government is going to look for possibilities to constitute groups of water treatment schemes that are together financially independent.
Schemes that are run by a District Council are financially organized differently. The District Council is financially supported by the Ministry of Local Governments. Employers get their salaries from this Ministry. Revenue that is collected goes to the Council. Running costs for the water supply scheme like energy, spare parts, chemicals etc. have to be financed by the Council.
In Chipata a private organisation runs the water supply scheme. The purpose of the private organisation is to keep the revenue that is collected within this water supply scheme in order to finance all running costs of the scheme. The large investments, like the dams to create reservoirs, cannot be financed with this money and is subsidized by the German Development Bank.



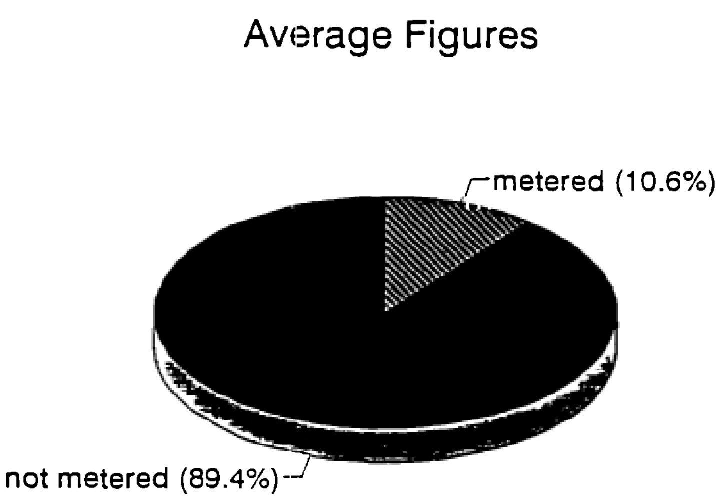


figure 3.3: Metered connections

Training

Most operators that work at the township water supply schemes have done a two years course for Water Plant Operator in Lusaka.
The education of the officers in charge of the various township water supply schemes deviates, some did a three year course Water Engineering in Lusaka, some did the two years course for water Plant Operator and some did no special education for this job.
In Chipata there has been a training-course for both officers and operators, where people that were going to run the plants in Chipata and in Solwezi, were trained on the job.

3.2 Analysis

Operation and maintenance

The preventive maintenance of the constructions at the schemes is mostly neglected.
When the defects are getting visible, they can already be mayor defects and then the repairs might be very expensive.
Spare parts and chemicals are very often not in stock are the place where they are needed. This is not only a problem of lack of money. Since it is too expensive to store all kinds of spare parts and chemicals at every scheme, the Department of Water Affairs has a central point in every Province where both chemicals as spares are stocked. However, right now it is not clear whether the needed materials are available and how long it will take to get them to the right place. Furthermore the lack of chemicals at a plant is recurrently noticed too late.
Due to the lack of small spares like parts of mains, sockets and water meters, that are regularly needed for repairs, the plumbers or other people concerned with the repair, have to make shift with the materials there are. As a result many leakages are repaired as good as possible with inappropriate materials. Badly leaking mains, for instance, are replaced by mains of other sizes. Consequently after a while it is unclear what sizes the mains in the reticulation have, and this makes it hard to calculate the capacity of the system. Another effect of this "creative plumbing" is that at places where the size of the mains suddenly changes, the velocity of the water flow changes and mains can easily get blocked etc.

* Chemicals are frequently not in stock at the water supply schemes. This is either caused by bad management, when the run out of the chemical is noticed too late, or by the availability of the chemical. It might be impossible to get the chemical in Zambia. The time of delivery can increase when a chemical has to come from abroad.
When chlorine is not available, water is not disinfected and water borne deceases like cholera can spread easily. This is a serious danger for the common health of the inhabitants of the township concerned.
It is obvious that it is of great importance that the stock of chemicals should be kept up more accurate to avoid these kind of problems.

Chemicals have to be stored carefully. Whether this is done all right at the townships that have been visited is not clear.
Big consumers like institutions often pay a fixed amount of money since their water consumption is not metered. It is most likely that the water consumption of a lot of these consumers is much higher.

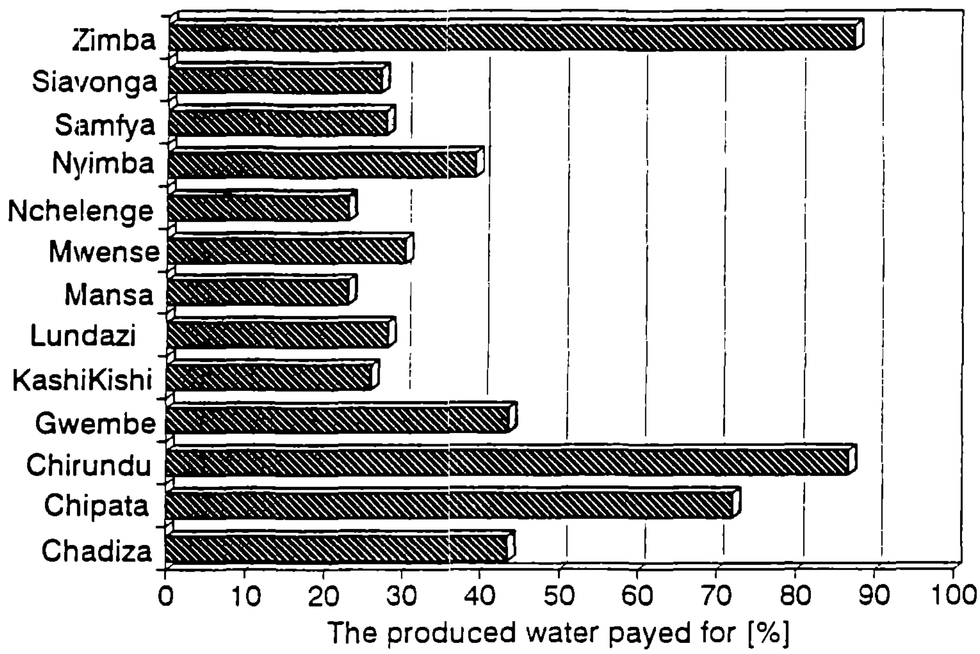
* Generally the supply of the produced water is irregular.
Since the production is insufficient to supply the consumers for 24 hours a day,there is water running from the taps at only certain hours. This rationing can cause rust and recontaminations in the mains (air might get in when the mains are empty) and the consumers are uncertain of the supply.
The missing of good maps of the area near a water supply scheme makes it difficult to find the exact place of the mains. The sizes of the distribution system are not well known so it is impossible to calculate the capacity of the distribution system. Due to the missing of good maps the differences in heights are not known. These are needed for instance to calculate whether a water hammer device is needed or not.

Management

Decisions that are made at management level do not always reach the people that are responsible for handling these decisions.
Schemes that are the responsibility of either the Department of Water Affairs or a District Council can not use the revenue they collect from their costumers to finance running costs of their own plant. Revenue and obligations are not linked.
In Chipata, where the water supply scheme is in hands of a private organisation, costs that have to be made to run the scheme and to pay the employers can be refunded by the yield of the revenue collection.
When the revenue that is collected at schemes, of both the Department of Water Affairs as the District Councils, is going to be used to finance the running costs of the water supply schemes, the motivation of the revenue collector might increase and at the same time the amount of money that is collected.

Training

One of the main problems of the water supply in Zambia, is a lack of well trained operators to keep the water supply schemes running properly.
Operators often do not know why they are doing what they are doing. There are standard procedures that can be performed as long as the plant is working under the average circumstances. This means that when the plant is not working under normal conditions, people in charge are not always able to find proper solutions.



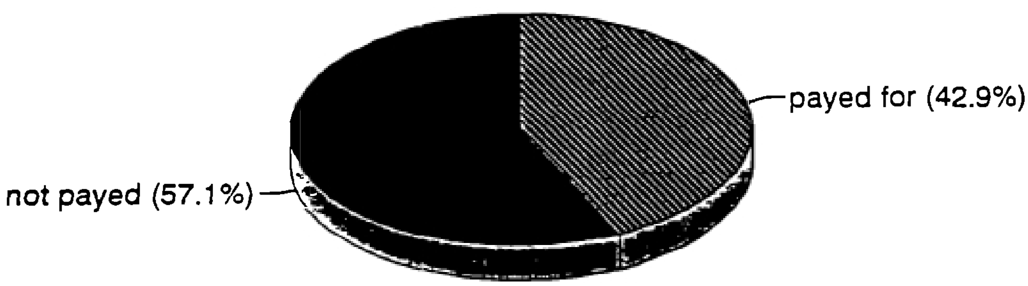


figure 3.4: Revenue efficiency

The lack of well trained operators might be caused by a shortage of people with some basic education that are willing or capable to do a training for plant operator. Another cause might be that the existing training courses do not link to the needs. More than in other townships in Zambia, operators in Chipata, who have done a special on the job training, seemed to know what they were doing and were able to find proper solutions when part of the plant was not functioning optimal.
Additionally there is a lack of people at higher level (engineers) that have a good understanding in the treatment processes and water supply techniques.

Table 4.1: Dates of building and responsible organization.

Dates	Chadiza	Chipata	Chirundu	Gwembe	Kashi Kishi	Lundazi	Mansa	Mwense	Nchelenge	Nyimba	Samfya	Siavonga	Zimba
Organisation	D.W.A.	other	D.W.A.	D.W.A.	D.C.	D.W.A.	D.C.	D.W A.	D W.A.	D W.A.	D.W.A.	other	D W.A.
approx.date of building	1976	1988	1960	1957	1968	1971	1976	1970	1960	1970	1986	1960	?
Rehabilitated		1988		1984									1985

D W A Department of Water Affairs, Ministry of Agriculture and Water Development
D C . District Council
others Chipata, semi private company, Siavonga, Kariba North Bank Company

Table 4.2: Intake comparison.

Intake of raw water	Chadiza	Chipata	Chirundu	Gwembe	Kashi Kishi	Lundazi	Mansa	Mwense	Nchelenge	Nyimba	Samfya	Siavonga	Zimba
Source	reser-voir	reser-voir	river	reser-voir	lake	river[1]	river	river	lake	reser-voir	lake	lake	reser-voir
Maximum[2] withdrawal	?	+	+	?	+	–	?	–	+	–	+	+	+
Determine[3] max.wtdrw.	yes	no	no	yes	no	yes	yes	yes	no	yes	no	no	no
Intake device	+	+	–	+	0	–	–	–	0	+	+	–	+[4]
Standby pump	yes	yes	yes	no	no	no	yes	no	no	yes	yes	yes	yes

+ satisfactory
– unsatisfactory
0 provisional, vulnerable
? capacity not known

1 The reservoir dam here has been washed away in 1986
2 This is compared with the demand based on a connection of 100% of the population to the water supply system
3 This is whether a good determination of the maximum withdrawal from the source is necessary or not
4 Zimba has two raw water reservoirs, one with a deteriorated intake device and one with a very good intake device

4 RESULTS OF SYSTEM ANALYSES

4.1 Introduction

The basis for the results presented in this chapter are the system analysis carried out for the thirteen water supply systems that are visited, which are given in appendix C. In these analyses the design and operation of the different water supply systems is per township compared with the design considerations given at the start of appendix C.
The results are given for the different components of the water supply system, starting with a short description of the types that are used in the visited townships, following with the present state and the design, and ending with the operation of the systems.
This chapter ends with a comparison between capacity of the systems and the water demand. The water demand calculation is given in appendix B.
The results and conclusions given here are illustrated by the tables on the left pages.

4.2 Intake of raw surface water

Description

Raw surface water is drawn from rivers, lakes and man made reservoirs. Mainly there are two types of intake structures.
First, a solid construction which is protected very well, with the possibility to abstract water, by gravity or with pumps, at several depths. For example intake towers and dams belong to this group. This type of intake device/construction has only been built in the raw water reservoirs and is used for abstraction from lakes or rivers(the new intake in Samfya is an exception on this).
Second, the more vulnerable and provisional structures. Usually these are built up by a GI (Galvanized Iron) intake main, going over the river bank or shore of the lake from the intake device to the raw water pumps. The intake device often only consists of a strainer at the end of the intake main. This type of structure is found at raw water abstractions from rivers and lakes.

Present state and design

The solid intake structures such as intake towers are still working well or at least reasonably well, even with low water levels in an extreme dry year as 1992.
The more vulnerable and provisional structures used for intake from rivers and lakes are often deteriorated and placed too close to the river bank or shore of the lake. The water quality close to the shore can be bad due to activity of humans, animals and the growing of for example algae. Also there is danger for flooding and erosion. Water levels can drop very far during the dry season, especially in the rivers, necessating the extensionof the mains and the replacemetn of intake devices.

Table 4.3: Comparison of the floc forming and removal.

Pre-Treatment	Chadiza	Chipata	Chirundu	Gwembe	Kashi Kishi	Lundazi	Mansa	Mwense	Nchelenge	Nyimba	Samfya	Siavonga	Zimba
Source	reserv.	reserv.	river	reserv.	lake	river	river	river	lake	reserv.	lake	lake	reserv.
Coagulant dosing	rainy season	year round	rainy season	rainy season	not at all	rainy season	n a.	rainy season	n a	rainy season	n a	rainy season	year round
Jar test	–	+	–	–	–	–	n a.	–	n a	–	n.a.	–	–
Visual control	–	+	+	+	–	·	n a.	–	n a	–	n a	–	+
Dosing system	+	+	–	+	–	–	n a	–	n a	+	n a	–	+
Rapid mixing	+	+	+	+	–		n a	+	n a.	+	n a	+	+
Floccu-lation	--	+	+	+	–	–	n a	+	n a	–	n a	+	+
Settlingtank													
inlet zone	+	+	+	+	–	+		+		+		+	+
settle.zone	+	+	+	–	–	+	n a.	+	n a.	+	n a	+	+
outlet zone	–	+	+	+	–	–		+		–		+	–
S_0 plain GWD100%[1]	+	–	–	–	–	–	n a	–	n a	+	n a	+	+
S_0 flocculent GWD100%[1]	+	+	+	+	+	+	n.a.	+	n a	+	n a	+	+
Overall operation	–	+	–[2]	0	–	–	n a.	–[2]	n a	–	n.a	–[2]	0

+ executed or works satisfactory
-- not executed or works unsatisfactory
0 reasonable
* indicates a problem

1 Comparison between the s_0 for the Gross 100% connection Demand(also appendix B) and the allowable s_0
2 It is unknown that the sedimentation tank works with a sludge blanket

The present situation is that these intake devices only consist a tube with a strainer, just thrown into the water near the river bank. Hence a relatively bad water quality is obtained. Sludge, biological activity and pollution cause this relatively bad water quality.

Operation

In almost every visited township it is unknown what the exact capacity of the source and pumps is. For lakes and large rivers the relatively small abstractions for the water supply have nearly no influence, but for small rivers and reservoirs the maximum withdrawal is very important. A lot of figures are lost and reservoir capacities are said to decrease due to silting. When also the amount of abstracted water is not measured, it is always a guess if there is water left at the end of the dry season.
Also, information about static heads over the raw water mains and pump characteristics is often missing. That is why sometimes pumps and mains are not geared to one another.

4.3 Surface water treatment

4.3.1 Floc forming and removal

Description

In most of the townships the treatment is equipped with pre treatment by floc forming and removal.
Aluminium salt, 'alum', is commonly used as coagulant and dosed mainly during the rainy season when the turbidity of the raw water is high. Flocculation is carried out sometimes in special flocculation chambers, sometimes in a combined flocculation and sedimentation tank and in 30 % of the cases not at all. Two different types of sedimentation tanks are used. Most commonly used is the rectangular, horizontal flow, settling basin and in three cases a combined upflow sludge blanket reactor was found.

Present state and design

The design of the systems is not always adequate.
The coagulant dosing devices are at 50 % of the systems provisional or even not existing. The other relatively good working devices are often inaccurate.
In a few cases there is no proper rapid mixing of the coagulant possible and three systems even do not have flocculation facilities. Sedimentation tanks, except for the outlet zone, are still in good condition. The outlet zone often collects the water in one point instead of over (more than) the whole width, causing eddies and short circuiting.

Table 4.4: Comparison of sand filters and disinfection.

Filtration Disinfection	Chadiza	Chipata	Chirundu	Gwembe	Kashi Kishi	Lundazi	Mansa	Mwense	Nchelenge	Nyimba	Samfya	Siavonga	Zimba
Sand filter	slow	rapid	rapid	slow	rapid	slow	slow	rapid	n a	slow	slow	rapid	slow
Filter run[1] in days	30	2 and 5	3	14	1	?	3	2	n a	20	10	0 25	7
Filter rate[2] in m/h	0 12	4 6 and 5 3	2 2	0 16	2 2	0 08	0 17	1 6	n a.	0 07	0.15	4 2	0 09
Turbidity[2] removal, %	96	66	-13	60	15	83	75	4	n a	92	40	69	54
Sand d_{10} U thickness	-- -- +	+ -- +	-- -- ?	+ 0 --	-- -- ?	+ -- +	 + +	0 -- *	n a	-- + +	0 + +	0 + *	+ 0
Backwash rate	n a	+	--	n a	--	n a	n a	--	n a	n a	n a	--	n a
Cleaning procedure	--	+	--	--	--		--	--	n a	--	--	--	--
Control	--	+	--	--	--	--	--	--	n a	--	--	--	--
Overall operation	--	+		--	--	--	--	--	n a	--		--	
Disinfection dosage ppm	-- 0 3	-- 0 5	-- 0 6	-- 1 1	-- 0 9	-- 0 6	-- 0 2	+ 3 0	-- 1 7	-- 1 2	-- 0 7	-- 1.4	+ 2 85
Dosing device	--	--	--	+	--	--	+	+	+	--	+	--	--
Mixing	--	0	--	--	--	--		--	--	--	--	--	--
Time of contact	+	+	+	+	+	+	+	+	+	+	+	+	+

+ satisfactory
-- unsatisfactory
0 reasonable
* indicates a problem

1 An estimation, gathered from conversations with operators
2 Figures for the dry season with good raw water qualities. The turbidity removal is a random indication but the filter rate is an average over at least one month (estimated from production and filtering hours)

Operation

Operation of the floc forming and removal process is very bad in nearly all townships. Most important for the operation is the coagulant dosing. To determine the optimal coagulant dosage and pH, regular Jar tests should be carried out, which in practise is only done in Chipata(in all other townships operators are not familiar with this test and there is no jar test apparatus available). The second way to determine and adjust the dosing is by visual control of the raw and treated water. This is done in some townships, but in a very irregular and inaccurate way. Also the dosing devices that are good, are often treated wrong, for example a alum solution for one day dosing is prepared in the morning in the dosing device but then the whole day clear water is added to the device causing too extreme dilution of the solution and too low concentrations. Adjustment of the pH, by for example a continues lime dosage, in order to create better floc forming conditions, also is only done in Chipata. The flocculation and sedimentation tanks usually only need regular cleaning and are operated in a reasonable manner, the problem being that the efficiency is influenced by the coagulation process which is often not very good. The upflow sludge blanket tanks are not very well operated. At the three townships where these tanks were placed operators had no idea that the tank was designed for floc removal by a sludge blanket. That is why sludge removal is done in a completlye wrong way, by draining the complete tank and by manually removing the sludge sometimes, instead by a regular draw off of a part of the sludge blanket.

4.3.2 Filtration

Description

Sand filtration is in all townships the last treatment before disinfection. Both rapid as slow sand filters are commonly used. Most of them are designed as inlet controlled filters with an overflow weir in the effluent line. Only a few are outlet controlled with an adjustable valve or V-notch to change the resistance over the outlet or with an automatic floater system. The backwash facilities for the rapid filters mostly consist of an elevated reservoir which is filled with water from the clear water tank by a small pump.

Present state and design

The design of the filters is usually acceptable to good and most of them are relatively simple to operate. Overflow weirs placed at approximately the same level as the surface of the sand bed prevent negative pressures to occur. Only the rapid filters in Chipata do not have such a weir. The few automatic floater systems that are placed have often broken down and are controlled manually.
Some townships have more or less deteriorated constructions, especially Mansa, but here the filters are already under rehabilitation.

Table 4.5: Capacity comparison with the gross 100% demand.

Capacity[1] comparison	Chadiza	Chipata	Chirundu	Gwembe	Kashi Kishi	Lundazi	Mansa	Mwense	Nchelenge	Nyimba	Samfya	Siavonga	Zimba
Source	?	+	+	?	+	–	?	–	+	–	+	+	+
Raw water pump	+	–	–	+	?	+	–	–	–	–	+	+	+
Raw water main	+	?*	–	+	–	–	–	+	–	+	+	+	+
Sedimentation	+	–	+	+	+[1]	+	n a	+	n a	+	n a	+	+
Filtration	+	+	+	+	+[1]	+		+	n a	+	+	+	+
Clear water tank	+	+	+	+	+	+	–	+	n a	+	–	+	+
Clear water pump	+	–	–	–	?		–	–	n a	–	–	n a	–
Clear water main	+	+	–	+	–	+	–	+	n a	+	+	n a	–
Service reservoirs[3]	+	?*	–	+	–	–	–	–	–	+	+	+	+
Distribution	?*	?*	–	–	–	–	–			–	–	–	–

\+ equal or higher capacity
– lower capacity
? capacity not known
?* *missing figures*

1 The capacities are only possible if everything is maintained and operated very well
1 Construction is deteriorated so badly that it is doubtfull if it can be upgraded much
3 Fire fighting and supply during breakdowns are not taken into account

Worst are the sand layers. in at least two townships the sand layer in the slow sand filters was not more than 20 cm thick, and almost all filters have more or less deviating effective grain sizes and coefficients of uniformity. Except for Chipata, in all townships that use rapid filters, the backwash rates are too low, causing accumulation of dirt in the sand bed. The low backwash rates are the result of too small backwash reservoirs with too small drains or backwash pipes, or when direct backwashing with pumps is applied, the capacity of the pumps is too low. Air scour is only applied in Chipata, while it is necessary in more townships while many use relatively small sand grains.
The underdrainage and filter bottom has not been looked at.

Operation

One of the problems with the filter operation are the sand characteristics. For slow sand filters the effects of slightly differing parameters as effective grain size and uniformity are not too striking or disastrous, but for some rapid filters the effects are more obvious. Rapid filters often use sand that has not sufficient uniformity, causing grading during backwash and consequently cake filtration, filter cracks and mud balls. The result is short filter runs lower effluent qualities. Also, the effective size of the grains, d_{10} is sometimes relatively small which makes is hard to produce enough scour during backwash in order to clean the grains thoroughly, causing accumulation of dirt in the sand bed. With all slow sand filters the cleaning procedure causes deterioration of the active biological layer, while with every cleaning the filter bed is drained and dried completely before the scraping starts. This drying kills the active organisms, and while it takes about one or two months to build up a new biological layer, which is more then the time between two successive cleanings, biological treatment will never adequate. Also the sand layer scraped off with every cleaning is often too thin, which causes accumulation of dirt in the sand bed.
Filter control is in most cases not carried out as it is designed, mostly causing declining rate filtration, with high filter rates just after cleaning, instead of constant rate filtration. During the rainy season rapid clogging of the filters is reported at nearly all treatments. This rapid clogging is mainly caused by the often failing pre treatment, thus giving high turbidity loads on the filters.
The total operation of the filters and the pre treatment causes too short filter runs often low filter rates. The effluent quality is not biologically safe, but the turbidity is, despite bad operation, for nearly all filters (except two rapid filters) relatively low. A note here is that the visits were all in the dry season when the turbidity of the raw surface water is also relatively low.

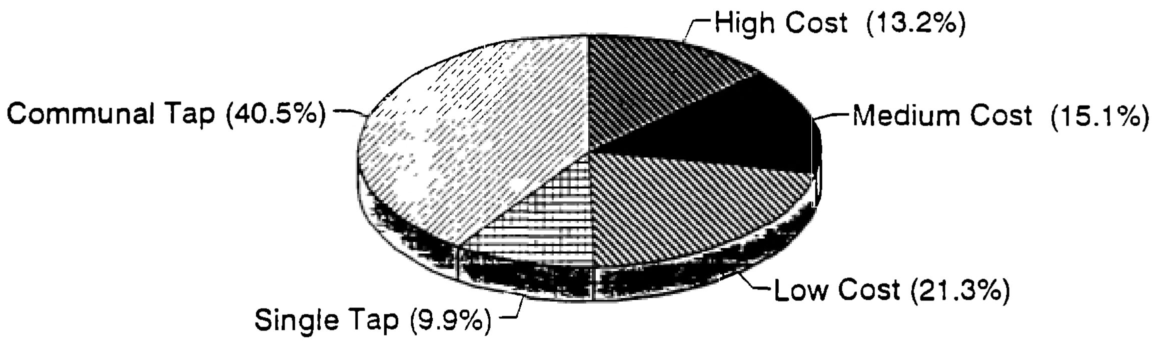


figure 4.1: Average connection division over the different types of housing.

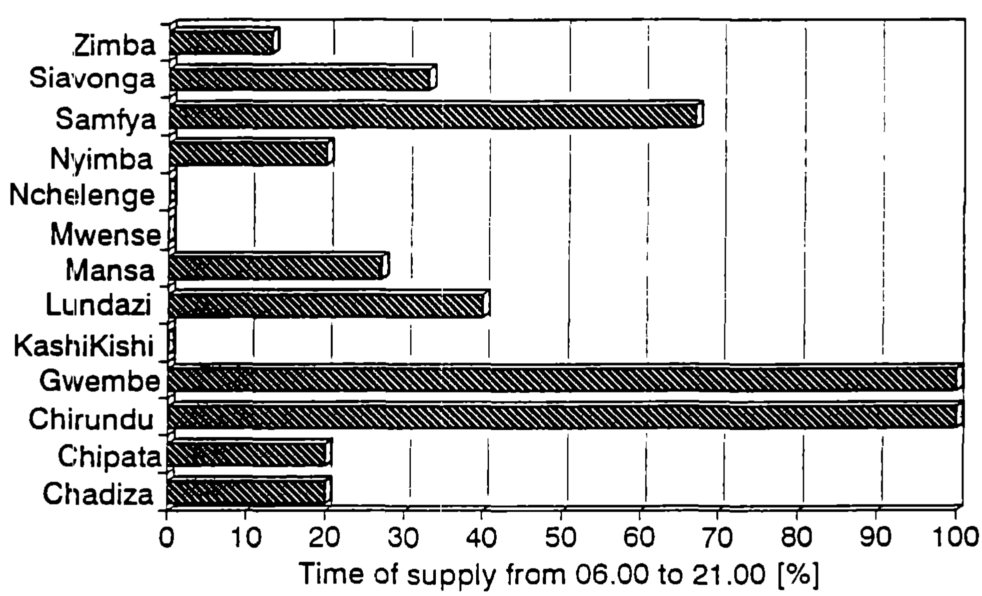


figure 4.2: Water supply in problem areas.

4.3.3 Disinfection

Description

HTH (High Test Hypochlorite) powder is used as disinfectant in nearly all townships and it is dosed straight into the clear water tank.

Present state and design

There is not much left of the original design, most dosing devices have been replaced or removed. At two places old gas chlorinators were found but these were out of operation due to lack of gas chlorine.
At many treatments no proper dosing device for HTH is available.

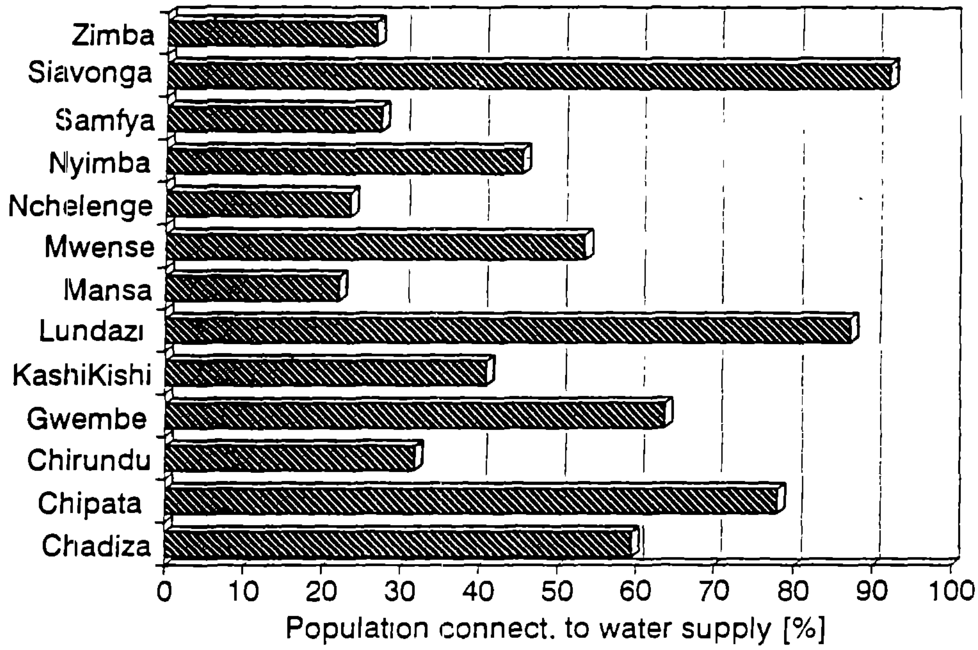
Operation

HTH powder is often one or a few times a day just dumped into the clear water tank. No proper distribution mixing can take place in this way. Only at a few places continuous dosing occurs with a constant concentration.
Moreover, the average amount of HTH dosed in most of the townships is too low for a thorough disinfection of the treated water, and often there is even no HTH powder available for some time. The time of contact is long enough at all sites.

4.4 Storage and distribution

Description

The first storage is always in the clear water tank. In most townships this tank has the function to store the differences in production of the treatment and the pumping of the clear water pumps to the service reservoirs and sometimes the supply of the backwash water. It also creates contact time for the disinfectant.
The townships are nearly all supplied by gravity from elevated service reservoirs or ground reservoirs at a high point near the township. Transport of clear water is in most cases done by pumping it directly through a clear water main from the clear water tank to the service reservoirs. There are both private connections as well as public stand posts, see also figure 4.1.



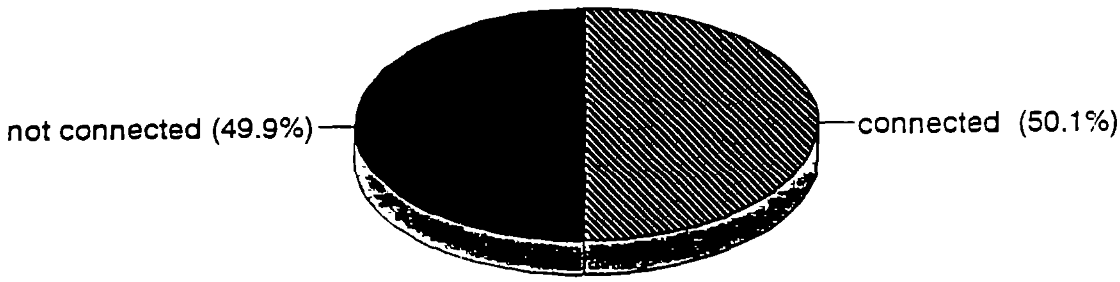


figure 4.3: Connected population

Present state and design

The design as well as the present state of the clear water reservoirs and service reservoirs is usually good. Only some reservoirs were slightly deteriorated and leaking.
The distribution system is worse. There are almost no proper maps with elevations, distances or sizes, which makes it impossible to calculate pressures; this also goes for raw and clear water mains. Water hammer provisions are only placed in Chipata and Samfya. The one in Samfya is not working and in other townships it is not known if such a provision is required. Many leakages and lack of spare parts are reported. Also in every township there are some compounds that get no water even though they are connected or only get water for a few hours a day, see figure 4.2. In most cases this has two causes, first the water production is too low for the demand and second, the capacity of the distribution system is too low for the demand. The second one causes that people in some parts of the system do not get even at times the service reservoirs are full.
The percentage of the population that is connected to the water supply system is often very low, as shown in figure 4.3, so a lot of illegal use of water can be expected.

Operation

If not enough water is produced, water is often rationed, especially during the night, reducing the water losses. Leakages are normally reported by consumers and the revenue collector should check every house regularly. Problems are concentrated in official government buildings where problems occur regarding responsibility. Repairing of leakages is often done provisionally; due to lack of spare parts. Water meters are placed at only a few percent of the connections which makes it hard to check if standardized tariffs are fair for everyone.
Expansions of the network are done in most cases without any proper design, thus asking for trouble with the distribution.

4.5 System capacity

In table 4.5, the maximum possible capacity of the water supply systems is compared with the gross demand per township if 100 % of the population of that township is connected to the water supply system (for calculation see appendix B). An important conclusion here is that for nearly all the systems the capacity of the treatment components as well as the capacity of the clear water tank are sufficient. The capacity of pumps, mains and distribution system is insufficient in most cases.

5 CONCLUSIONS AND RECOMMENDATIONS

With design and rehabilitation of water supply systems in Zambia, an integrated approach of the following aspects is indispensable.

Design and applied techniques.

In the design of the water supply systems, regular errors occur. It is doubtful if the systems, with a surface water treatment and distribution network, as used in many townships, are sustainable and economically feasible.
Ground water or induced recharge near rivers (bank filtration) could be alternative sources for surface water. If not, the techniques available for clarifying surface water should be carefully weighed on criteria as economical and operational feasibility. The pre treatment commonly used in townships, floc forming and sedimentation, falls short of operational feasibility at the moment. For example roughing filtration could be a better option.
A thorough study is necessary to show in what way township water supply is feasible. Aspects as 'willingness to pay' and operational preconditions are important here.

Operation

The overall operation of the water supply systems is very bad, which makes the guarantee of supply low. Main reasons for this are:
- choice of technology and design errors,
- operators do not have enough insight in the working of the system,
- there is almost no good backstopping and supervision, among others due to lack of qualified engineers,
- lack of good job descriptions causes pushing off ones responsibilities,
- lack of requisites, ranging from pumps to laboratories.

Better educated engineers, with good insight in the water supply systems, can give proper backstopping and on the job training of operators. Cooperation with the UNZA (University of Zambia) could be fruitful here, in the way of delivering trainers, and backstopping for engineers.

Management

As there is no direct link between revenues collected from consumers and money received for operation and maintenance, motivation to work cost-effective is absent.
There is a lack of qualified people: engineers as well as others that work in the water supply.

Maintenance

Present maintenance is merely concentrated on highly necessary repairs. Preventive maintenance could save a lot of trouble and unnecessary deterioration of the water supply systems.

For example rehabilitation of an existing pre treatment system or device, without looking at alternative systems or devices, the training of operators and backstopping, has no use while often the same problems as before will occur again, sooner or later, due to the fact that not all the causes have been considered and corrected.

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In Zambia we were permitted to use some measurement equipment from both the Technical University in Delft and the University of Zambia.
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APPENDICES

APPENDIX A

INVENTORY OF TOWNSHIP WATER SUPPLY IN ZAMBIA

1 CHECKLIST

General

* date of building
* electricity source, problems
* operation, which hours
* production, are there bukmeters
* future expansions, provisions
* measuring equipment, needed chemicals
* measurements, data

Intake

* capacity of source, high and low levels, base flow of the river and name
* sate of dam, intake, spillway, seapage
* present storage, how many month
* silting problems
* raw water, pumped or by gravity
* which pumps, how many working, Ns, H, Q, spare parts
* which hours abstracting water, Q per day, per month
* flow measured, how
* size of intake main, material, diametre, length and head
* air valves, wash outs
* intake: depth, construction, screens

Sedimentation

* size of tank: L, B, H, how many tanks available, working
* alum dosed?, when, how, how much and why, dosing system
* coagulation, flocculation, G-value(mixing) etc.
* coagulant storage, how many kg, when does shortage occur
* sludge removal, how often, drains
* lime dosed?, when, how much and why
* turbidity measured?
* Jar tests

Filtration

* how many filters, how many working at the same time
* type of filters
* size and sand layer thickness
* how often cleaned/backwashed and how
* sand monster
* filter rate measuring, drop down measurements
* how many hours a day working
* how are they operated
* clogging problems, how is this noticed

* do the filters overflow sometimes
* is clear water directly used after cleaning/backwash
* backwash rate, water/air, how long
* available head loss
* underdrainage, material, thickness
* filter control, type, is it working, consequences

Clear water tank
* dimensions
* disinfection: how, how much and why, dosing device
* disinfectant storage in kg, when do shortages occur
* time of contact: reservoir capacity and pumping rate

Clear water pumps
* how many, Ns, H, Q, spare parts
* preventive maintenance
* when and which hours are they pumping, and to where
* water hammer provision
* rising main, size, material, leakages, head, bulk meter
* air valves, wash outs, non return valve

Distribution
* spare parts, pipes, meters etc. available
* leakages
* reservoirs: where and capacity
* what reservoir serves what part of town which hours
* big consumers: e.g. secondary school how many pupils etc.
* new connections possible?
* size of big(first) sevice mains, material
* plumber checks taps?, how often, police etc.
* when is there water in town; high parts

Water quality
* turbidity: intake, after sedimentation, filtration, tap
* pH: intake, after dosing, tap
* temperature, intake and tap
* conductivity: clear water
* alkalinity: intake and tap
* colour: after filtration, on sight
* residual chlorine: clear water tank and tap
* algaes

2 INVENTORY

2.1 CHADIZA (1976)

GENERAL
* DWA is in charge of the water plant and supply system
* electricity from hydro power plant Katete
  breakdown twice a week

INTAKE
* intake from Nsadzu river; reservoir
* during heavy rains of 1978 spillway is broken
* two raw water pumps, both working
  pump 1 (1990): Q = 62 m^3/h (measured by operators);
  used at night time: 18.00-8.00; starts itself
  pump 2 (1976): Q = 91 m^3/h; used during day: 8.00-18.00
* the capacity of the pumps is checked last year (by timing
  the filling of sedimentation tank)
* no spare parts in Chadiza; there are some in Chipata
* small breakdowns are fixed by operators
* intake main is good (no leakages), 500 m length

SEDIMENTATION
* two tanks each 3 * 12 m; height 2.8 m to 4.0 m (bottom has slope)
* rapid mixing in baffle channel
* no proper flocculation
* alum feeder is not working
* 10 kg alum a day is dosed, only in rainy season, in 24 hours
  added to tank continuously
* high turbidity in rainy season
* two times a year tank is drained and sludge removed
* after dry season: 0.2 m sludge in dividing chamber at beginning
  of sedimentation tank
  after rainy season: 0.7 m sludge is found
* drains are said to be OK

FILTRATION
* four slow sand filters, A = 176 m^2 each
* in rainy season: each filter is cleaned once every four weeks
* in dry season each filter is cleaned every twelve weeks
* filters were overflowing at time of visit
* cleaning: they dry a filter completely, then upper layer is removed
* one filter is empty, there will come new sand in august.

* new sand goes is after five years, 0.8 m is left then after refilling 1.0 m of sand in the filter
* upper part of the old sand is removed, rest is used again after cleaning in sand washing bay
* in rainy effluent season water is coloured

CLEAR WATER TANK

* capacity 454 m^3
* chlorine is used the whole year in clear water tank
* chlorinator does not function since two years
* 250 g HTH per day is added at 5.00 h in the reservoir, reservoir is filled with water, than water is pumped to reservoirs in town

CLEAR WATER PUMPS

* normally there are two pumps for the township supply centrifugal Ritz pumps 2975 r/min, Q = 90 m^3/h; 8.5 hours a day working one has broke down since last year
* when both pumps are working there is maintenance
* one mental pump for hospital and water plant mental Ritz pump Q = 30 m^3/h, centrifugal, n = 2910 r/min; 5 hours a day working
* operating one can be repaired when it breaks down minor repairs in a few hours, for mayor repairs spares have to come from Chipata

RETICULATION

* clear water main: 225mm AC, $\Delta H = 128$ m, length is 11.5 km
    * no spare parts
    * replacing rubbers is mayor repair
    * six air valves, some are badly leaking
    * 5 wash outs, all are working and used once in 6 month and after repairs
    * some airvalves are closed to stop leakages
    * three reservoirs
        1. 454 m^3
        2. 340 m^3
        3. 163 m^3 for secondary school
    * reservoir 1, in township is leaking badly. It has been reported to Chipata several times but no response
    * reservoir 1 and 2 are connected
    * water supply to town
      5.00-9.00 hours
      11.30-14.00 hours
      18.00-21.00 hours

* new houses on higher grounds; worst neighbourhood:
  12.00-13.00 water
  18.00-20.00 water
* every day a plumber goes round
* system is very old GI, AC, PVC pipes
* no spare parts, "creative plumbing"
* water meters enough but no fittings
* during school holidays no complains about water quantity

WATER QUALITY

* pH and residual chlorine were checked but not any more since they run out of tablets
* at time of visit:
    * intake
        * turbidity 65.7 ntu
        * temperature 18.9 °C
        * alkalinity 1.2 mmol/l
    * after sedimentation
        * turbidity 42.4 ntu
        * after filtration
        * turbidity 1.94 ntu

PRODUCTION

raw water pump:	a.62 m^3/h (1990)
	b.91 m^3/h (1976)
clear water pump:	a.90 m^3/h
mental pump :	30 m^3/h

a. bulk meter:	07/06/92	Q = 969 m^3
	08/06/92	Q = 782 m^3 (4 hours no electricity)
	09/06/92	Q = 1.019 m^3
	10/06/92	Q = 1.099 m^3

February '92	: 31.367 m^3
March	: 26.486 m^3
April	: 33.295 m^3
May	: 33.328 m^3

average is 31.119 m^3 --> 1.000 m^3/day

STORAGE

high ground reservoirs:		a. 454 m^3
		b. 340 m^3
secondary school res. :		164 m^3
		---
	total	958 m^3 = 96 % of the daily production

clear water reservoir :	454 m^3 = 45 % of the daily production
intake storage :	? m^3 = ? month production

2.2 CHIPATA

GENERAL
* Two systems
* electricity normally 24 hours a day supply
  January-March '92 problems with supply

LUTEMBWE I

INTAKE
* from lutembwe river
* reservoir 1.5 million m^3
* silt in reservoir because of erosion. illegal farming
  at hill sides
* dam ok
* 1.25 m below normal level now (9 June 92)
* intake tower full of water
* two intake levels
* normally gravity to treatment works
  if level is very low, pumps are used
* no flow measurement
* two raw water pumps, both working
* spare parts available

SEDIMENTATION
* circular tank, accelator type, approximately 11 m
  diameter, with an inner circle of approximately 5,80 m
  diameter. Settle area(outer circle): 69 m^2
* alum is dosed by feeder; 30 min on/30 min off
* two alum feeders available (one spare)
* sludge washed out automatically every 12 minutes by
  opening sludge valve for a few seconds
* lime dosed in rainy season with alum feeder

Filtration
* four rapid filters, 2.52 * 4.00 m each
* stickpendel of one of the filters broke down
* average 0.8 m of sand in filter
* two types of sand; 1:1 mixed
* floaters that should control supernatant level are not
  working

* three times a week back washing of all filters in three hours
* during backwashing the raw water pumps are in use to higher the supernatant water on the other filters in order to produce the same amount of water for the town
* backwashing procedure: * air blowing for 30 minutes
  * backwashing with water & air for 15 minutes
* two air blowers, both working (one is spare)
* during dry season of '79, '91 and '92 no water production for 8 hours a day
* four rate flow meters
* average flow 45 m^3 / hour for each filter

* after backwashing, water that is filtered immediately goes in clear water tank

CLEAR WATER TANK
* normally gas chlorination, machine is still working
* since 1990 gas is not available; HTH powder used
* every two hours 200 g HTH
* protection against water hammer

CLEAR WATER PUMPS
* two clear water pumps (1 is spare), both working
* 6 days a week pump 1 works
  1 day a week pump 2 works, pump 1 is checked
* total production 4360 m^3/day

RETICULATION
* spare parts are at Lutembwe 2
* 17 bar pressure on main pipe

WATER QUALITY
* intake
  * turbidity 14.2 ntu
  * alkalinity 2.1 mmol/l
* after sedimentation
  * turbidity 7.89 ntu
  * alkalinity 1.8 mmol/l
  * Temperature 19.7 °C
* after filtration
  * turbidity 2.72 ntu
  * residual chlorine 0.3 mg/l

LUTEMBWE II

INTAKE
* 6 million m^3 in reservoir
* dam in lutembwe river, 7 km from other dam
* 24 h a day water is taken in
* intake tower is dry
* spill way stopped spilling end of March
* two raw water pumps (one is spare), both working
* capacity 208 m^3/h; 5000 m^3/d; peak hour 247 m^3/h
* raw water balancing tank 250 m^3
* water level after dam is checked (for seepage)
* silt spur from tower (tunnel through dam)

SEDIMENTATION
* 209 m^3 tank
* in rainy season alum added and mixed
* three times a year tank is cleaned
* instructions for alum dosing after lab test
* original pipes for alum and lime to flush mixer are clogged; bypass is used

FILTRATION
* two rapid filters, 18 m^2 each
* back wash tank 100 m^3
* every five days both filters are backwashed

CLEAR WATER TANK
* capacity 450 m^3
* chlorination every hour 200 g HTH powder
* two water hammer tanks after clear water tank

CLEAR WATER PUMPS
* three clear water pumps; one at the time used for five days
* $Q = 190 \, m^3/h = 4550 \, m^3/day$

2.3 CHIRUNDU (1960)

GENERAL
* electricity from main net; no problems with delivery
* run by DWA

INTAKE
* direct from Zambezi river
* always enough water
* two raw water pumps; both working
    * 30 hp; used in dry season
    * 20 hp; used in rainy season
* operating hours 05.00 - 22.00 h
* intake main 75 mm GI, length 500 m; head is unknown
* no air valves, no wash outs
* intake is pipe in river with screen in front of opening

SEDIMENTATION
* one sedimentation tank: steel plates 4.50 m diameter, H = 1.2 m; cone 4.25/2.0 m diameter, H = 2 m
* 3.40 m water in tank
* 4 kg alum dosed a day, at once
* when the water is dirty, more alum is dosed, just on sight
* 90 kg alum in stock, no shortages
* monthly the amount of chemicals is reported
* no lime dosed

FILTRATION
* one rapid filter, 2.75 m diameter; $A = 5.94\ m^2$, $H = 3.2$ m
* one meter of sand in filter
* backwashing once a week; in rainy season twice a week
* two backwash tanks, 9 m^3 each, full when overflowing
* backwashing until water is clean, no air added
* clogging of filter is noticed by the water level in the filter
* design filter rate = 30 m^3/h = 5 m/h
* height from backwash tanks to filter bottom is 4 to 5 m
* filters overflow after one day in rainy season every week in dry season
* after backwashing filter is directly used
* available head loss is approximately 1 m
* underdrainage: crashed stones 0.5 m, big stones 0.25 m, Than sand

CLEAR WATER TANK
* capacity 68.3 m^3
* chlorination 200 grams/ day HTH powder added continuously
* this amount of chlorine is dosed on advice of the provincial water engineer
* at 6.00 h dosing tank is filled, during day water is added so the solution dilutes
* structure ok

CLEAR WATER PUMPS
* two clear water pumps, one 7.5 hp pump with Q is approximately 13 m^3/h (calculated from production) and a small pump, standby, Q is not known
* pumping hours: 5.00 - 22.00 h
* January '92 last breakdown, 1 day to get spares from Choma
* preventive maintenance on pumps since february (new reporting system DWA)
* clear water main 75 mm AC, length approximately 500 m
* one non return valve
* no washouts, no air valves
* bulk meter is there

RETICULATION
* spare parts: 5 water meters and 250 mm AC pipe at time of visit these spares were not needed
* they say that there are no leakages
* one reservoir, 68.3 m^3; half full at time of visit, height 15 m
* they say that new connections possible but transport for pipes is necessary than
* big service main: 75 mm AC pipe
* water plant operator is plumber; taps are said to be checked daily
* during operation there is said to be water on every tap

WATER QUALITY
* no measurement equipment available at plant
* water quality at time of visit:
    * intake
        * turbidity 1.85 ntu
        * alkalinity 0.8 mmol/l
        * temperature 24 °C

* after sedimentation
    * turbidity 1,59 ntu
* clear water tank
    * turbidity 1.75 ntu
    * no residual chlorine (14.00 h)
* tap at compound
    * temperature 26.2 °C
    * turbidity 10.5 ntu

PRODUCTION

bulk meter: average production for January, April and May:
$Q = 6600$ m^3/month, thus 220 m^3/day

STORAGE

clearwater tank: 68 m^3 = 31 % of the daily production

elevated reservoir: 68 m^3 = 31 % of the daily production

intake storage: Zambezi river

2.4 GWEMBE

(1957, rehabilitated in '84)

GENERAL
* in charge of DWA
* electricity from Hydropower station Livingstone during dry season no problems; during rainy season once in two months no electricity for two days

INTAKE
* two intakes from Chikuni river
    * Singonya dam, good condition
    * Gwembe dam, seepage and much silt
* Gwembe dam: two pumps, both broken since 4 months; broken parts are already made, only payments have to be done
  $Q = 25\ m^3/h$; $n = 1450$ r/min; $H = 11.25$ m
* Singonya dam: two pumps, one working
  $Q = 51\ m^3/h$; $H = 82.5$ m; $n = 2900$ r/min
  'standby' generator since '87 but no diesel
* operation from 4.00 till 20.00 hours
* non return valve in main to plant
* 2 bulk meters, after both raw water pumps one
* intake main 150 mm AC, length 4000 m
* one main from intake to treatment plant; only one pump working at the time
* 3 airvalves, all working
* no wash-outs, capacity of main is good
* intake by foot valves with a wall around it

SEDIMENTATION
* receiving chamber
* two tanks, built after '79, height 3.2 m; width 2.25m; length 5.65 m
* alum dosed in rainy season only: average 5 kg (depending on reaction they vary this amount) solved in water every morning, added by dripping during the day continuously
* rapid mixing at beginning of baffle channel
* baffle channels for flocculation
* water level controlled by floaters
* no alum in stock, they try to get it to save it for rainy season
* in rainy season sometimes there is no alum available, lime is never dosed
* jar test can not be done

FILTRATION

* three slow sand filters, two small ones, which work at the same time, and one large one
* one large or two small filters are working at the time
* operation 24 hours a day
* large filter: diameter 11.2 m
  small filters: diameter 7.9 m
* the small filters are designed for a filtration rate of 0.15 m/hour
* they intend to get new sand for filters from a nearby stream
* once a month filters are cleaned in dry season by removing a small layer; once in two weeks in rainy season
* V-notch after filters
* large filter: 15 cm of sand (measured), gravel can be seen between sand
* small filters are said to have a sand layer of 60 cm
* filters never overflow; gate valves are closed when water rises
* after cleaning, water goes directly in clear water tank
* filter control: floater connected to gate broken; now done by hand gate can be opened when necessary

CLEAR WATER TANK

* built after 1979
* length 10.26 m; width 4.31 m; height 2.48 m
  capacity 110 m^3
* chlorine dosing system; 600 g a day in dry season and 1 kg in rainy season is added continuously
* 50 kg HTH in stock; no shortages occur

CLEAR WATER PUMPS

* two pumps,both working
  $n = 2900$ r/min; $Q = 27.6\ m^3/h$; $H = 29.6$ m
* spare parts come from Choma, prov.water engineer
* pumps are generally checked once a week
* clear water main 150 mm AC; length approximately 900 m.
* no leakages
* bulk meter in clear water main
* one non-return valve in main
* no wash-outs, airvalves after bulk meter

RETICULATION
* no spare parts available; they come from Choma
* water meters in stock
* only reported problems are checked by a plumber
* once a month the revenue collector checks taps and reports
* two reservoirs in town, total capacity 250 m^3, no leakages
* big consumers: hospital (not metered), Genery (metered), primary school (not metered)
* big service mains 100 mm diameter
* no complains about water shortages in higher parts

WATER QUALITY
* no measurement equipment at plant
* at time of visit:
    * intake
        * turbidity 33.6 ntu
        * temperature 18.5 °C
        * alkalinity 1.2 mmol/litre
    * after sedimentation
        * turbidity 32.4 ntu
    * after filtration through small filters
        * turbidity 13.1 ntu
    * tap at high compound
        * alkalinity 1.2 mmol/l
        * turbidity 20.3 ntu
        * trace of chlorine (less than 0.2 mg/l)

PRODUCTION

bulk meter gives a production of 11620 m^3/month
388 m^3/day

STORAGE
* Clear water tanks 110 m^3 = 28 % of daily production
* reservoirs in town 250 m^3 = 64 % of daily production
* intake storage at Gwembe Dam is unknown
  at Singonya dam is unknown

2.5 KASHI KISHI (1968)

GENERAL
* in charge of Council
* electricity supplied by Zesco, problems in rainy season

INTAKE
* intake from lake Mweru
* intake pipe 40 to 50 m long, 4 m deep
* strainer in front of foot valve
* 1 raw water pump, n= 2950 r/min, Q is unknown
* intake 24 hours a day
* intake main 150 mm GI
* no non return valves

SEDIMENTATION
* 1 tank, 8.4 m diameter, 3.5 m depth; 190 m^3
* no alum dosed for last 15 years; no money
* tank is not straight, overflow at one side only
* sludge removed once a year with drain valve

FILTRATION
* two rapid sand filters, 2.4 * 3.0 m each
* back wash pumps are not working since at least last month
* normally backwashing every two days in dry season in rainy season once a day
* sand in overflow, waste of sand
* no flow meter
* filter rarely overflows
* backwashing for one hour with both air and water
* after backwashing water goes straight to clear water tank
* underdrainage: graduated stones

CLEAR WATER TANK
* 465 M^3
* until 10 years ago they used gas chlorination now HTH powder is used when available; 1 kg a day directly in clear water tank
* in case of a cholera outbreak they beg at various places to get HTH powder (DWA, Lusaka, health institutions)
* no storage of chlorine

CLEAR WATER PUMPS

* one pump, 1987, Q is unknown, n = 2950 r/min
* no spares, no oil etc for preventive maintenance
* distribution to secondary school from 21.00 to 6.00 h
  other hours distribution to village
* clear water main to sec. school reservoir 150 mm AC
* 3 airvalves, one wash out in clear water main, all working
* no water hammer provision (broke down)

RETICULATION

* no spare parts
* secondary school reservoir 280 m^3
  2000 pupils, 53 staff houses, 35 houses
* big consumers are sec. school and milling company, both not metered
* new connections are possible when new consumers buy there own pipes etc
* size big service mains: 150mm; 100mm, 50mm
* 3 plumbers
* leakages in house have to be reported by owners themselves
  or by revenue collector
* high part of town: chandwe: whole day water

WATER QUALITY

* a comparator to measure residual chlorine is available at plant but no chemicals to use it
* after sedimentation
    * turbidity 5.39 ntu
* water at tap
    * alkalinity 0.8 mmol/l
    * T = 22.5 °C
    * turbidity 4.58 ntu

PRODUCTION

* if the secondary school reservoir is filled completely from 21.00 h till 6.00 h, then the water production is 22400 m^3/month

STORAGE

* 1 reservoir 280 m^3
* 1 clear water tank 465 m^3

2.6 LUNDAZI (1971)

GENERAL
* in charge of DWA
* since March '92 the electricity supply is good (new generator)

INTAKE
* in '86 the dam (1946) is washed away; until then there was enough water
* now every dry season sandbags are put in the river to rise the water level; this does not work very well (seepage)
* 10 km upstream there is a dam and a reservoir; when dam at Lundazi is improved with sandbags, this dam will be opened so that temporary a reservoir at lundazi will be created
* three raw water pumps, only one working two are broken down in '86, not back yet
* flow meter is not working

SEDIMENTATION
* no coagulation/ flocculation tank; alum directly added to sedimentation tank
* no baffles
* alum dosing device is broken down
* alum is only dosed in rainy season
* sedimentation tank cleaned twice a year
* 4 bags of 50 kg alum each in stock

FILTRATION
* 4 slow sand filters, A = 176 m^2
* a red line on the filter will indicates the filters need to be refilled
* after cleaning of filter the water that is led through the filter immediately goes to the clear water tank

CLEAR WATER TANK
* capacity 454 m^3
* chlorinator is broken since aug '91
* 300 g HTH directly in water tank twice a day dosing at 14.00 hours; at 16.05 hours chlorine concentration less than 0.2
* three drums of HTH powder in stock

CLEAR WATER PUMPS

* three pumps; only one is working('77)
  one broke down in 1988, one since may '92
* pumping from 24.00-10.30 hours and from 14.00-19.00
* small leakages in clear water main
* clear water main 200 mm AC pipe; H = 125.84 m + height of tank above main; length = 1.8 km
* clear water main is leaking
* no air valves, no wash outs

RETICULATION

* two reservoirs, filled at the same time and connected at bottoms
* 140 m^3 and 87 m^3 capacity
* pipe from reservoirs is 80 mm AC
* third reservoir at secondary school (1400 students) has a capacity of 315 m^3; height 3.91 m
  filled form 0.00-5.00 h and two hours in afternoon
* main to secondary school is 150 mm
* ground level difference between secondary school reservoir and treatment is 42.35 meter
* service mains AC pipes; normally 2 feet under ground
  rainy season: erosion, not enough ground to protect pipes, pipes get broken by cars
* GI pipes: oxidation of pipes
* leakages: broken pipes: 'creative plumbing'
* high leaking percentage
* water from secondary school tank: straight into big main to shops in the lower parts, no working valves, so water runs trough to lower parts, control is not really possible.
* two times a week there is no water in town
* secondary school has no water from 12.00-15.00 h according to DWA Lundazi
* compound near secondary school has no water from 10.00-17.00 hours and after 19.00 hours
* people of compound do not boil their water before use

WATER QUALITY

* no measurements for water quality are done at plant
* intake
  * turbidity 18.1 ntu
* after sedimentation
  * turbidity 4.15 ntu
  * alkalinity 2.8 mmol/l

* after filtration
    * turbidity 0.69 ntu
    * alkalinity 2.8 mmol/l
* at tap
    * no residual chlorine

PRODUCTION

a. estimated themselves: 21.000 m^3/month
b. slow sand filter discharge, monday 8th June:
2 filters each 176 m^2 surface
Q1: 23,0 m^3/h, 7,4 cm in 34 minutes
Q2: 5,6 m^3/h, 1,8 cm in 34 minutes
total 28,6 m^3/h, 20.592 m^3/month
c. secondary school reservoir in 5 hours approx. 250 m^3 --> 50 m^3/h, 23.350 m^3/month, as the high pumps work only from 0.00-10.30 h and from 14.00-19.00 h

Q = 21.000 m^3/month, 700 m^3/day seems realistic, since not more can go through the slow sand filters.

STORAGE

elevated reservoirs:	a.	140 m^3
	b.	87 m^3
secondary school res.:		315 m^3
		---
total		542 m^3 = 77 % of the daily production

clear water reservoir: 454 m^3 = 65 % of the daily production
intake storage reservoir: 0 m^3

2.7 MANSA (1976)

GENERAL
* in charge of District Council
* electricity supplied by Zesco
  problems during rainy season 3 to 4 times a month

INTAKE
* intake from Mansa river
  normally enough water, this year there will be problems
* small weir of stones and sand bags to rise water level
* 3 submersible pumps
    * 1 flight pump; Q = 192 m^3/h
    * 2 ploiger pumps; Q = 150 m^3/h
* one of the pumps is not placed very solid, the intake main is leaking and the intake construction is bad
* 2 other pumps are in Copperbelt to be repaired
* operation pumps 24 hours a day
* no flow measurements
* intake main 150 and 200 mm; 3 intake mains

COAGULATION/ FLOCCULATION/ SEDIMENTATION
* not done

FILTRATION
* 4 slow sand filters, 54 * 10 m each; normally two filters in operation
* now in one of the filters underdrainage and sand is replaced
* sand layer thickness 1 meter; 300 mm sand above drain pipes
* underdrainage by porous drain pipes in sands
* every three days the filters are cleaned
* filters get new sand every 4 years
* clogging problems in rainy season: water level in filters rises and no discharge
* outflow by gravity
* cracks in filter walls
* water is led after filtration in open receiving chamber (was supposed to have a lid)
* filters overflow often in rainy season
* lime dosed normally every day in receiving chamber; 500 g/hour

CLEAR WATER TANK
* capacity 1138 m^3
* 500 g chlorine (HTH) added at least at each shift (4 shifts a day) depending on amount of produced water dripping in clear water tank
* 10 kg of HTH in stock at time of visit

CLEAR WATER PUMPS
* three pumps; two broken (one since 1 month, one since 5 months)
  1 working: Q = 270 m^3/h; H = 85 m; n = 1490 r/min
* pumping 24 hours a day
* non return valves on pumps are broken; they say water hammer can be heard when there is a power failure
* no air valves, no wash outs
* clear water main 300 mm GI; length 1.2 km

RETICULATION
* no spare parts available
* many leakages in reticulation
* two reservoirs
* 1 of 1138 m^3 for whole town
    * 1 of 250 m^3 for hospital
* big consumers have water meters but they are broken down
* distribution from 6.00 h till 18.00 hours
* two plumbers check daily the reticulation
* in highest part of town (Mansa Batteries) there is water between 6 -10.00 hours

WATER QUALITY
* equipment to measure residual chlorine and pH is available at plant but since 1 month they run out of tablets
* in rainy season the turbidity is high; clear water is coloured in beginning of rainy season for approximately one month
* intake
    * turbidity 4.8 ntu
    * alkalinity 0.3 mmol/l
* filtered water
    * turbidity 1.2 ntu
* in clear water tank
    * turbidity 3.4 ntu (part of the water is not filtered)
* in clear water main
    * residual chlorine 0.45 mg/l

PRODUCTION

* normally 10000 m^3/day, depending on quality of filters and working hours of pumps (their estimation)
* now, calculated 194400 m^3/month (clear water pump capacity)

STORAGE

* reservoirs
    * town reservoir 1138 m^3
    * hospital reservoir 250 m^3

      1388 m^3 = 21% of daily

* clear water tank 1138 m^3 = 18% of daily production

2.8 MWENSE (1970)

GENERAL
* in charge of DWA
* electricity supplied by Zesco, problems in rainy season

INTAKE
* intake from Mwense stream, basic flow is 12 l/s; until this year no problems with water quantity
* no dam, small weir to rise water level
* two raw water pumps; 1 broken since feb.'92
  1 flight pump from Nchelenge; Q = 220 m^3/h
  1 KSB pump ; Q = 15 m^3/h in use from 18.00-5.00 h
* now a borrowed pump is used
* no spare parts
* intake main 150 mm AC; 900 m length
* water is led to receiving tank, 3.2 m diameter, H = 3.2 m where little fish are swimming

SEDIMENTATION
* 2 tanks, steel plates 4.5 m diameter, H = 2 m
  cone 4.25/2.0 m diameter, H = 1.2 m
* discharge channels leading to filters are very dirty; full of rust and weeds
* alum dosed in rainy season 10 kg a day
  at 6.00 h 5 kg is added at once in receiving chamber
  at 14.00 h 5 kg is added at once in receiving chamber
* in dry season alum is rarely added
* no alum in stock
* in rainy season they get 50 kg alum every month from provincial water engineer in Mansa so 25 days a month they can not add alum
* every day sludge is removed by valve; once in 6 months they clean sedimentation tank completely
* lime is never dosed

FILTRATION
* 2 rapid filters, diameter 2.75 m, H = 3.20 m
* design filter rate 5 m/h
* once a week back washed in dry season in rainy season once in two days or when flow rate goes down and clear water is dirty
* 30 minutes backwashing using back wash tanks
* 4 backwash tanks, 9 m^3 each; bottom approximately 8 m above sand surface.
* underdrainage consists of graduated stones

CLEAR WATER TANK

* capacity 450 m^3, underground storage reservoir (concrete, 16.8 m diameter, H = 2.10 m) for gravity supply to the major part of the consumers
* leakages and cracks in wall of tank
* 500 g HTH is dosed in 100 litre water; 2 kg added continuously in 4 to 5 hours
* 20 kg HTH in stock; they get 50 kg a month which is enough for 25 days; since it is a cholera area the Ministry of Health gives chlorine for the other 5 days
* They say that if they do not have chlorine, people are advised to boil the water before drinking

CLEAR WATER PUMPS

* 1 pump working; 1 pump broke down and is gone Q = 24 m^3/h
* no preventive maintenance; they say operators are not skilled to do it
* when taps that get water from elevated tanks run out of water, they start pumping again unless there is not enough water in clear water tank
* non return valves after pump as water hammer provision
* clear water main to elevated tanks 150 mm GI , no leakages
* no airvalves, no wash outs in clear water main

RETICULATION

* no spare parts at all, only in Mansa people are told to buy parts that have to be replaced themselves; receipt will be refunded
* leakages can not be repaired due to lack of spares
* supply to town from 6.00-9.00 h; from 11.00 -14.00 h and from 17.00 to 20.00h
* elevated tanks 24 m^3 total
* elevated tanks are empty after 6 hours; open from 6.00 to 12.00 h and from 13.30 to 19.30 h
* new connections are not possible because of lack of pipes
* big service mains 150 mm
* 1 plumber and two helpers check public taps daily

WATER QUALITY

* no measurement equipment for water quality at plant
* intake
    * alkalinity 0.5 mmol/l
    * temperature 19 °C
    * turbidity 16.0 ntu

* after flocculation
    * turbidity 16.7 ntu
* after sedimentation
    * turbidity 17.8 ntu
* in clear water tank
    * turbidity 17.0 ntu
* at tap
    * no chlorine
    * turbidity 14.7 ntu

PRODUCTION

* average over last four months: Q = 14030 m^3/month (calculated by checking capacity of clear water pump every month)

STORAGE

* reservoir 345 m^3 = 74 % of average daily production
* clear water tank 450 m^3 = 96 % of average daily production

2.9 NCHELENGE (early 60th)

GENERAL
* in charge of DWA
* electricity supplied by Zesco, problems during rainy season at least once a week

INTAKE
* intake from lake Mweru
* pipe goes into the lake, coarse strainer in front of opening (fine strainer cloggs)
  when waterlevel goes down, the intake pipe is extended
* no silt problems
* 1 low lift pump working, Q = 33 m3/h; H = 41 m, n = 2900 r/min
* 1 standby diesel set but pump is not connected because there is no fuel
* no spare parts for pump
* intake main 75 mm AC/GI, 80 m length
  suction into lake 75 mm GI, 15 m length, 1 meter deep
* no airvalves, no washouts

NO SEDIMENTATION

NO FILTRATION

WATER TANK
* capacity 112 m3
* tank is cleaned every two months
* tank is leaking badly
* 1.08 kg HTH is added by continios dripping in 24 hours into clear water tank
* 1 drum of HTH in stock

HIGH LIFT PUMPS
* 1 pump, Q =? , n = 2900 r/ min; 10 HP
* no preventive maintanance (no oil etc)
* from 18.00 h to 6.00 h they pump to elevated reservoirs
* 2 non return valves as water hammer provision
* rising main 50 mm PVC/AC, length ca 1000 m
* 2 wash outs, used once a year when a lot of mud comes out

RETICULATION

* almost no spares
* water meters are there but not installed because they get broken by sand etc in the water
* many leakages, serious onces are repaired when spares from Mansa arive
* reservoirs
    * police: 12 m3, filled in 4 hours
    * FTC: 18 m3 filled in 6 hours
* no big consumers
* new connections are not possible because the capacity of the pumps and lack of pipes
* there is one plumber that repairs leakages, reported by consumers, when there are spare parts
* big service main is 50 mm GI
* in high parts there is water inly at night 22.00- 24.00 h

WATER QUALITY

* when it is windy the intake water is very dirty
* no equipment to measure the water quality
* at compount
    * turbidity 4.86 ntu
    * alkalinity 0.8 mmol/l
    * small trace of chlorine
    * temperature 23 C

2.10 NYIMBA (1970)

GENERAL

* in charge of DWA

INTAKE

* intake from Chikuyu river/dam
* reservoir 155.600 m^3; the capacity is less now because of silt
* two raw water pumps; 1450 r/min; H =15,8 m; Q = 30 m^3/h
* operation from 14.00 -24.00 h

SEDIMENTATION TANK

* two rectangular tanks; one is leaking badly and not in use
* 4 kg of alum a day is dissolved in a container while water is added continuously (during time concentration of solution gets less)

FILTRATION

* two slow sand filters, A = 176 m^2 each; one used at the time
* filters are cleaned after three weeks; then no water comes through the filter any more: overflow due to clogging
* filters can run dry
* underdrainage gradually increasing stones

CLEAR WATER TANK

* capacity 454 m^3, circular tank
* HTH powder added 500 g a day; mixing every two hours; water is added continue, concentration of solution decreases during day
* limited stock of chlorine

CLEAR WATER PUMPS

* two pumps working; H = 36 m, n = 2900 r/min; Q = 31 m^3/h.
* operation 9.5 hours a day
* clear water main 150 mm AC, 500 m length

RETICULATION
- * no spare pipes
- * no serious leakages
- * reservoirs:
  - * four small tanks closed at top, 36 m^3 total, filled in 1.5 hours
  - * one ground reservoir, 454 m^3 with open windows; three hours pumping to this reservoir a day; it is half full then
  - * secondary school reservoir 210 m^3, five hours pumping to this reservoir
- * water from 4 tanks: 6.00- 8.00 hours
  17.00-20.00 hours
- * water from ground reservoir: 6.00-12.00 hours
- * many complaints about water supply
- * taps checked every month
- * water in higher part of town from 6.00 -8.00 h and from 17.00-18.00 h
- * big service mains 3 * 80 mm

WATER QUALITY
- * intake
  - * turbidity 9.9 ntu
  - * alkalinity 5.3 mmol/l
- * at tap
  - * turbidity 0.77 ntu
  - * no residual chlorine

PRODUCTION

2 clear water pumps: 516 l/min = 31 m^3/h, 36 metre, n = 2900 r/min

a. 9,5 hours pumping per day = 294 m^3/day = 8.820 m^3/month

5 hours secondary school res.: 155 m^3
4,5 hours township reservoirs : 140 m^3

STORAGE

high ground reservoir : 454 m^3
elevated 4 drums* 9 m^3: 36 m^3
secondary school res. : 210 m^3
---
total 700 m^3 = 238 % of the daily production

clear water reservoir : 454 m^3 = 154 % of the daily production
Chikuyu dam reservoir: 155.600 m^3 = 17,6 @ month production

@ because of silting and reed growth is the reservoir capacity reduced. How much is not known at the moment.

2.11 SAMFYA (1986)

GENERAL
* Run by DWA
* electricity supply by Zesco

INTAKE
* from lake Bangweulu
* intake by two suction pipes 200 mm
* strainer to prevent weeds to get in
* in the rainy season intake water is very dirty
* 3 raw water pumps; Q = 90 m^3/h 1986 (all working); H = 8 m, n = 1450 r/min
* normally one pumps at the time is used, sometimes two pumps are used together for 1 hour
* operation hours 24 h a day
* intake main 200 mm GI, length 300 m
* 3 airvalves, checked once a month
* no wash outs

NO SEDIMENTATION

FILTRATION
* 8 slow sand filters 18 * 6 m each
* bypass pipes in the filter bed are installed to increase water production
* average filter rate 0.15 m/h
* dry season 4 at the same time
  rainy season 7 at the same time
* cleaning in dry season once a month; in rainy season once every two weeks
* only dirt on top of sand is removed, no sand
* filter dried completely before cleaning in dry season
* operating by filling the filter from the bottom and overflow for a while to get rid of last floating dirt
* clogging problems in rainy season
* they say the filters are cleaned when less than 20 m^3/h (0.19 m/h) comes through; a discharge of 50 m^3/h (0.46m/h) for 1 filter is good
* regular overflow of filters
* underdrainage 1.5 m including sand (approx. 1 m)
* every filter has an inlet valve; they try to get the same supernatant level in every filter by adjusting the valves
* hard upper sand layer (sand sticks together)

* outlet controlled filter

CLEAR WATER TANK
* 82 M^3
* chlorination by filling dosing device with 1.5 kg HTH at 6.00am, dripping 24 hours a day
* storage for 7 days now
sometimes they do not have chlorine for months

CLEAR WATER PUMPS
* 2 pumps, 70 m^3/h each (both working) 1986; H = 42 m, n = 500 r/min
* no spares; no tools for maintenance
* pressure tank and non return valve as water hammer provision
* pressure tank is not working (compressor is broken down)
* clear water main 200 mm GI; length 300 m
* no airvalves no washouts in clear water main
* 1 bulk meter in clear water main

RETICULATION
* no spare parts, they order when needed from provincial water engineer
* they say many leakages,due to lack of spares
* 2 elevated reservoirs 20 m^3 each type Braithwaite
2 sec school reservoir 20 m^3 and 90 m^3
1 ground reservoir 1100 m^3
1 ground reservoir 420 m^3
* 3 automatic Booster pumps to fill elevated reservoirs, H = 14 m; Q = 55 m^3/h, n = 2900 r/min
* supply hours to town
4.00- 14.00 h
16.00- 20.00 h
* 150 mm pipe from elevated reservoir,
200 mm pipe from ground reservoir
* bulk meters after reservoirs normally read every day; no stationary so they stopped to register Q
* big consumers : sec school (950 boarders, 50 teachers and 300 non-boarders) metered, small hospital, metered not working
* worst neighbourhood has water from 6.00-14.00h and from 18.00 -20.00 h

WATER QUALITY

* there is a residual chlorine comparator; reagents available
  tested 3 times a week, always chlorine at tap, >0.2 mg/l
* water quality at time of visit:
    * intake
        * temperature = 20 °C
        * turbidity 5.0 ntu
        * alkalinity 0.4 mmol/l
    * after filtration
        * turbidity 3.0 ntu
    * at public tap
        * turbidity 5.0 ntu
        * trace of residual chlorine

PRODUCTION

* average over three month: Q = 46563 m^3/ month (bulk meter)

STORAGE

* clear water tank 82 m^3 = 5 % of daily production

* 1 sec school reservoir (elevated)	20 m^3	
		90 m^3
* 2 elevated reservoirs 20 m^3 each	40 m^3	
* 1 ground reservoir		1100 m^3
* 1 ground reservoir		420 m^3

= 107 % of daily production		1670 m^3

2.12 SIAVONGA (approx.1960)

GENERAL

* last year the Kariba North Bank Company took over the plant from DWA reason: water production was too low (300 m3/day)
* no historical data available
* electricity from power plant siavonga, no breakdowns
* a new treatment unit is placed in order to expand the system
* supply from 05.00 till 21.00 hours, pumping and treatment 24 hours a day

INTAKE

* water level reservoir is so low that pump had to be lowered
* dam is good, not the responsibility of water plant (Kariba Dam)
* no silt problems, no odour/colour problems
* dead weeds obstruct the pumps and settle in tank
* 3 raw water pumps in stock; one 40 hp spare motor
* two lines of pumps working 24 hours a day: a 15 hp and a 60 hp pump on one line
  1 pump: HP 45(60); V 380/440; RPM 2920; G*D*D= 1.48 KGm2; Q = 75 m3/h; H = 90 m (1982)
  1 pump: D225M; 2930 RPM (1981) HP 60
  2 pumps: G*D*D = 255 KGm2, HP 15
* raw water main 100mm GI and 150 mm AC; length = 300 m, ΔH = 75 m;
* spare parts available
* no airvalves, no wash outs in rising main
* no screens in front of intake pipes with footvalves, small holes in footvalve; floating weeds block it.

SEDIMENTATION

* two tanks, steel plates diameter 4.5 m, H = 1.2 m
  cone diameter 4.25/2.0 m, H = 2.0 m partly under ground
* alum dosed 1 kg in the morning; one in the afternoon manual, in one tank only
* no alum available since nine months; not available in country
* sludge removed once a week by opening valve in tank
* no lime dosed

FILTRATION

* two rapid sand filters, both working, diameter 2.75 m; A = 5.94 m2
* thickness of sand layer is not known
* filters clog rapidly

* backwashing in rainy season 6 times a day
  in dry season 4 times a day
* two backwash tanks; 9 m3 each on roof
* backwash tanks are filled in 1.5 h (new pumps)
* backwashed only with water, not with air
* backwashing is stopped when water is clean
* after backwashing filters are directly used
* filter never overflows
* approximately 10 m from sand to backwash tanks
* when water level in tanks rise, filters are backwashed
* sedimentation and filtration drains are leaking

CLEAR WATER TANK
* two clear water tanks
  * 270 m3
  * 720 m3
* at 1 filter outlet: chlorination
  600 grams HTH per 6 hours, dripping continuously
* enough chlorine at other station, when necessary they get more

CLEAR WATER PUMPS
* no clear water pumps, plant is at highest part of township
* distribution by gravity
* bulk meter in supply line

RETICULATION
* Kariba North Bank Company buys spares, always enough
* reticulation is the responsibility of Council
* Council collects revenues
* there are 43 big consumers in siavonga
* taps are checked every week by a Plumber
* there are many extensions; some have problems now
  (government compound and zambia lodge area)
* hospital gets water from 10.00 to 13.00 h
  Zambia lodge area from 9.00 to 11.00 h and 18- 21.00 h
* many pipes above the ground; causes leakages
* in early days higher parts had water; now after many
  extensions not any more

WATER QUALITY
* no measuring equipment at the plant
* intake
  * turbidity 1.20 ntu
  * temperature 25 C
  * alkalinity 0.9 mmol/l

* after sedimentation
    * turbidity 1.28 ntu
* clear water tank
    * turbidity 0.42 ntu
    * 0.2 mg/l residual chlorine
* tap at compound
    * turbidity 0.5 ntu
    * temperature 26 C

PRODUCTION

average 1200 m3/day; bulkmeter

STORAGE

* no reservoirs
* clear water tanks * 270 m3
  * 720 m3
  ________ +
  total 990 m3 = 82.5 % of daily production
* intake lake Kariba

2.13 ZIMBA

(rehabilitated 1985)

GENERAL
* in charge of DWA
* electricity from Zesco power plant, in rainy season breakdown once a month for 4 hours

INTAKE
* intake at Nunukala river; Zimba dam:
  * capacity 91.058 m^3
  * state of dam good
  * seepage
  * spillway looks bad; clear water main through spillway
  * silt problems; less capacity in reservoir
  * gravity to pump house, pumping to treatment
  * 1 pump working: H = 84 m, Q = approximately 50 m^3/h *

real head is 50 m
  * intake main 150 mm GS, 200 m to pump; 2 km to treatment works
  * no wash outs, no airvalves
  * no bulk meter
  * intake normally 5.85 m under water level, now lower because system sank to bottom

* intake at Nasiankorgo river; Railway dam:
  * capacity 377.725 m^3
  * long spillway; erosion at the end
  * pumping to pump house, pumping to treatment
  * 2 pumps working H = 84 m, Q = 45 m^3/h
  * intake main 150 mm steel; length 4780 m; H = 57 m
  * 4 airvalves, all working; checked once in 6 months; 1 washout used once a year
  * enough storage to last dry period
  * no flow measurements; there is a meter but person in charge can not read

SEDIMENTATION
* receiving chamber
* baffle channel for flocculation
* 1 sedimentation tank, circular, diameter 12 m
* alum dosed in dry season 4 kg/ day; added continuously
  in rainy season 2 * 5 kg a day
* 5 kg alum in stock (!); they will report this tomorrow

this is one day storage
* sludge removed by wash-out
  in rainy season once in two months
  in dry season once in four months
* no lime dosed
* they lack equipment to do a jar test but are trained how to do it

FILTRATION
* 2 slow sand filters, working at same time; diameter 12m
* sand layer 15 cm and 20 cm approximately
* cleaning
  in rainy season once a week
  in dry season once a month depending on outflow
* clogging problem only in rainy season
* after cleaning the filters for approximately 3 hours they drain the produced clear water, if the colour of the produced water is good, they start filling the clear water tank
* filter overflows sometimes (once in two months)
* underdrainage stones (20 mm) layer of 15 cm

CLEAR WATER TANK
* capacity 100 m^3
* chlorine 2 kg HTH a day is added at once at 6.00
* dosing device is broken
* less than 50 kg HTH in stock, for 25 days

CLEAR WATER PUMPS
* only one pump, other is away since 3th June, new pump will come; $Q = 23.2\ m^3/h$ (measured)
* some preventive maintenance
* 2 storage tanks at treatment plant, from these tanks water goes to railway reservoir and sec. school reservoir by gravity
* pumping when water is available in clear water tank
* non return valve as water hammer provision
* clear water main 100 mm AC/GI, no leakages
* no airvalves, no washouts in clear water main

RETICULATION
* there are some spare parts: some meters for domestic houses and some GI pipes

* many leakages, no spares to repair them
* reservoirs
    * at treatment 2 tanks of 136 m^3 each
    * at secondary school 1 ground tank of 63 m^3 and 1 elevated tank of 15 m^3
    * 2 railway reservoirs of 254 and 58 m^3
* 2 tanks at treatment serve town
* main to railway 150 mm GI; to secondary school 75 mm AC and to hospital 50mm GI/AC
* revenue collector checkers taps in houses every month and reports leakages; repairing is responsibility of house owner; when tap is not maintained following month the house is disconnected
* rationing 6.00 -9.00 h
  12.00-14.00 h
  17.00-22.00 h

WATER QUALITY
* intake
    * turbidity 42.4 ntu
    * temperature 16.5 °C
    * alkalinity 0.3 mmol/l
* after sedimentation
    * turbidity 17.9 ntu
* clear water tank
    * no residual chlorine
    * turbidity 8.16 ntu

MANAGEMENT
* monthly report to Choma, provincial Water Engineer
* 4 operators
  1 officer in charge who did 2 years course in Lusaka and a building certificate

PRODUCTION

bulk meter: average for February/March/April: 14.731 m^3/month
increasing to 17.368 m^3 in April

Q = 14.731 m^3/month, or 491 m^3/day

STORAGE

clear water tank: 100 m^3 = 20 % of the daily production

2 elevated res. : 272 m^3 (at the treatment)
2 secondary school res: 78 m^3
2 railway res.: 312 m^3
---
662 m^3 = 135 % of the daily production

2 intake reservoirs: check report DWA,
* Brian Colquhoun Hugh O Donnel
Mr. Makwawa, at Head Quarters

APPENDIX B

WATER DEMAND

1 EXPLANATION

The explanation and method of calculating of the figures in this appendix is given below. First the number of connections for every town is derived from a local or central DWA office or District Council, specified for every type of housing and other buildings. The monthly consumption is standardized, but these figures are slightly corrected:

Assumptions:
- an average family consists of 6 persons
- 50 % of the high cost houses have servant quarters
- 30 % of the medium cost houses have servant quarters
- the average population growth rate over the last two years was 2.5%

Resulting domestic demand per family:

- high cost housing, 280 l/c/d (litre/capita/day) average number of persons (including servants): 9	75 m^3/month
- medium cost housing, 150 l/c/d average number of persons (including servants): 8	36 m^3/month
- low cost housing, 100 l/c/d average number of persons: 6	18 m^3/month
- single tap housing, 50 l/c/d average number of persons: 6	9 m^3/month
- communal taps, 30 l/c/d average number of persons: 6	5.4 m^3/month

Other water demand

- GRZ institutions (Government offices and the like)		230 m^3/month
- Parastatial, industrial and commercial connections		275 m^3/month
- Boarder schools:	boarders, 100 l/c/d	3 m^3/month
	non boarders, 30 l/c/d	1 m^3/month
	teacher families, as medium cost housing	36 m^3/month
- Extra demand, 10 l/c/d for public open space, playing fields		0.3 m^3/month

Explanation of the different figures.

Population: number of inhabitants per township, derived from the 1990 census, Central Statistics in Lusaka, and updated for the end of 1992 with an average growth rate of 2.5 percent per year over 1991 and 1992.
Connected population: the total population that is officially connected to the water supply system, based on the number of houses connected with an average occupation of 7 persons per house (including servant families).
Connection percentage: connected population devided by the total population of a township.

Net Water Demand (NWD): number of connections * specific water demand (including Extra demand)
Gross Water Demand (GWD): NWD * 1.4, this 40 % is to account for leakages and system use
Net Water Demand 100% (NWD100%): the demand when the complete population is connected to water supply system. This is the domestic demand of NWD * (1/connection percentage) plus the other water demand of NWD.
Gross Water Demand 100% (GWD100%): NWD100% * 1.4 to account for leakages and system use.

Production % GWD: (production/GWD) * 100%
Production% GWD100%: (production/GWD100%) * 100%

Billing efficiency: the percentage of the total water production that is billed for.
Collection efficiency: the percentage of the billings that is really collected.
Revenue efficiency: billing efficiency * collection efficiency.
Metered connections: percentage of the total number of connections that has a water meter.

WATER DEMAND SHEET

		STANDARDS		NUMBER	OF	CONNECTIONS	
	connection categories		consumption [m3/month]	CHADIZ	CHIPATA	CHIRUND	GWEMBE
NOT	high cost	housing	75	3	0	12	0
METERED	medium cost	housing	36	47	0	7	0
	low cost	housing	18	19	0	19	0
	single tap	housing	9	3	0	0	0
	communal tap	families	5.4	225	0	117	217
	GRZ	institutions	230	11	0	4	4
	parastatials	indust/comm	275	15	0	8	4
METERED	houses		36	66	0	1	88
	GRZ	institutions	230	10	0	0	6
	parastials	indus/comm	275	11	0	1	7
SCHOOLS	boarders		3	800	0	0	0
	day	students	1	400	0	0	0
	teacher	families	36	38	0	0	0
BIG	CONSUMERS	[m3/month]	0	0	0	0	0
EXTRA	DEMAND	POPULATION	0.3	4,728	56,451	3,427	3,354
PRODUCTION			[m3/month]	31,000	250,000	6,312	11,620
BILLED	WATER		[m3/month]	19,220	225,000	4,800	9,940
COLLECTION	EFFICIENCY		[%]	70	80	114	51
CONNECTED	POPULATION			2,807	44,000	1,092	2,135
CONNECTION	PERCENTAGE		[%]	59	78	32	64
NET	WATER	DEMAND	[m3/month]	23,443	160,000	6,585	10,671
GROSS	WATER	DEMAND	[m3/month]	32,821	224,000	9,219	14,939
NET	DEMAND	100 perc.	[m3/month]	28,402	205,276	11,207	13,149
GROSS	DEMAND	100 perc.	[m3/month]	39,762	287,387	15,690	18,408
PRODUCTION	DIVIDED BY	GWD	[%]	94	112	68	78
PRODUCTION	DIVIDED BY	GWD 100%	[%]	78	87	40	63
BILLING	EFFICIENCY		[%]	62	90	76	86
METERED	CONNECTIONS		[%]	21	ERR	1	31
REVENUE	EFFICIENCY		[%]	43	72	87	44

KASHI KISHI	LUNDAZI	MANSA	MWENSE	NCHE-LENGE	NYIMBA	SAMFYA	SIAVONG	ZIMBA
58	13	461	5	8	6	29	39	5
0	139	259	76	54	29	52	56	12
277	48	251	86	53	0	95	180	1
0	41	0	41	32	38	295	0	28
137	380	12	36	29	48	11	600	144
19	11	43	12	19	4	19	15	4
12	30	180	5	8	1	12	28	7
0	50	0	0	0	71	0	24	53
0	5	0	0	0	4	1	0	3
0	19	0	0	0	6	0	0	1
950	1,000	3,170	650	0	700	950	0	0
300	400	0	1,250	0	300	300	0	0
50	50	250	57	0	35	50	0	0
0	0	32,880	0	0	0	0	0	3,000
8,921	5,795	39,025	3,942	5,271	3,494	13,648	6,837	6,349
22,400	21,000	194,400	14,030	13,680	8,820	46,563	45,000	14,731
14,557	19,074	97,150	8,886	9,842	3,263	19,300	14,040	6,187
40	31	46	48	32	106	67	87	208
3,654	5,047	8,631	2,107	1,232	1,589	3,724	6,293	1,701
41	87	22	53	23	45	27	92	27
25,372	35,158	170,969	15,792	12,094	13,124	25,416	25,486	12,477
35,521	49,221	239,357	22,109	16,931	18,374	35,582	35,681	17,468
42,490	37,064	373,391	22,127	25,019	20,211	52,788	26,548	22,759
59,486	51,890	522,747	30,978	35,027	28,296	73,903	37,167	31,862
63	43	81	63	81	48	131	126	84
38	40	37	45	39	31	63	121	46
65	91	50	63	72	37	41	31	42
0	10	0	0	0	39	0	3	22
26	28	23	30	23	39	28	27	87

APPENDIX C

TOWNSHIP SYSTEM ANALYSES

1 DESIGN CONSIDERATIONS

1.1 INTRODUCTION

In this part general demands for water supply systems are given, but it is far from complete. It is more a sort of guidance for checking a water supply systems as generally used in Zambia, giving demands on design and performance in words and indicative figures. The analysis of the water supply systems per township further in this appendix have more or less the same structure.

1.2 INTAKE OF RAW WATER

The surface water source used is very determinant for the treatment system. The water quality during the year determines i.a. the required treatment. The water quantity determines the maximum withdrawal of raw water from the source, which at least should equal the maximum daily water demand at the end of the design period. Different sources of surface water are rivers, reservoirs and lakes.

Quantity
When water is directly taken in from a river, without any reservoir, the base flow in the dry season is decisive for the maximum withdrawal. To determine this maximum withdrawal, flow measurements over at least one whole year should be known, in order to have an indication of the base flow, and furthermore, downstream use of water should be taken into account.
When the base flow is too low to satisfy the demand, sometimes reservoirs have been built in order to store enough water during the rainy season for supply in the dry season. Very important is the water balance which determines the total amount of available water by comparing the discharge of the river and the demand during the year. Also seepage from the reservoir, evaporation of stored water, silting and downstream use of water should be taken into account. In practise, to judge an existing reservoir, you need a lot of data, which in many cases are not available. Very important is to determine the effective storage and maximum withdrawal. For this you need the present storage capacity (excluding dead storage) and the percentage of this storage that is filled during the rainy season. A problem is erosion of cultivated areas. Due to silting reservoir capacities can drop, but exactly how much is mostly not known. To be able to judge the reservoirs with a known original capacity we take into account a safety factor of 1.5 for silting, seepage and evaporation, suppose there is no basic flow during the dry season and furthermore assume the reservoir is completely filled during the rainy season. The reservoir then has to be large enough to store water for the seven month dry season.
When a large lake is used as water source no quantity problems are expected.
Very important is that the water abstraction is measured. For example this can be done by placing a bulk meter just after the raw water pumps in the raw water main.

Quality
The quality of directly abstracted river water mostly shows seasonal fluctuations. Especially during the rainy season turbid waters can occur since heavy rainfall causes surface runoff which transports silt into the river. The consequence is a considerably varying load of suspended solids on the treatment during the year and even during a day. Erosion of cultivated areas within the river basin can worsen this. Pre treatment, mostly a coagulation - flocculation -sedimentation process, which can handle the varying amount of suspended solids, is required. The river upstream of the intake should be checked on industrial and domestic pollution. In a reservoir, and to a lesser degree in a river, there is, to some extent, 'self purification'. Dependable on the 'retention time' of the water in the reservoir or river, silt can settle and biological processes can reduce organic matter. The fluctuation of the water quality of reservoir water is less then with a direct river intake and consequently the pre treatment is less complicated to operate. A lake can be seen as a large reservoir in which settling and biological processes have many time to clear the water. Thus lake water mostly has a constant quality without much suspended solids and in many cases doesn't require pre treatment as for example a coagulation - flocculation - sedimentation process.

Construction

Reservoir dam and banks. Very important is that the overflow of the dam is in good condition. With no proper overflow structure or device water just flows over the dam and can damage the foot and the crest of the dam as well. The dam will deteriorate which may result in sliding and ultimate destruction of the dam. Also the reservoir banks should be stable.

Intake device
* Important is that water can be abstracted at any time. Water levels in reservoirs and rivers can vary in time and can be extremely low at the end of the dry season. This means the actual intake device must be placed deep enough under the water level or be variable. Another reason to build an intake device at several depths for large reservoirs and lakes is that among others stratification causes water quality differences over the depth and the best water quality is not always on the same depth. For example the top layer can contain algae while light can still penetrate this layer and deep layers can be anaerobic.
* To prevent bed load entering the intake device, the device must be placed a few meters above the river bed, reservoir or lake bottom. Too much silt can damage the pumps and overload the treatment.
* The construction must be protected against damaging, e.g. against flooding.
* Through contact with humans and animals water contamination can occur near the banks. Therefore the intake device should not be placed directly along the bank.
* Some kind of strainer should be placed to prevent coarse material coming into the intake device. The dirt can block and damage pumps and mains.
* The capacity of the intake device should be large enough to cover the demand during the design period.

Pumps and mains
Raw water pumps and mains should be able to handle the maximum day demand including all water losses in the system.

1.3 PUMPS AND MAINS

The capacity of pumps and mains should be geared to one another and be determined by the water demand at the end of the design period. Raw water mains should be able to handle the maximum day demand, distribution mains have to be designed for a maximum hour on a maximum day. The clear water discharge out of the treatment must be the same as the raw water flow into the treatment or there must be storage in a clear water tank. To make the system reliable and to be able to do proper maintenance, stand-by pumps are required.
The working head of the pumps depends on total head loss over the main. The total head is built up by the static head over the main and the head loss due to friction during operation: $H_t = H_s + H_f$. Losses over valves and bendings are not accounted for here, so the calculation is more accurate for long mains then short ones of only a few hundred metres. For calculations of the head loss due to friction the following formulae are used:

$$\Delta H = \lambda * \frac{L}{\phi} * \frac{v^2}{2 * g} \qquad \text{Darcy-Weisbach (1)}$$

$$\lambda = \frac{0{,}25}{\log[3{,}7 * \frac{\phi}{k_N}]^2} \qquad \text{Nikuradse (2)}$$

ΔH:	loss of pressure due to friction in	[m]
λ :	coefficient for friction	
L :	length of the main in	[m]
ϕ :	pipe diameter in	[m]
v :	velocity in the main	[m/s]
g :	gravitational acceleration	[m/s^2]
k_N:	equivalent sand coarseness according to Nikuradse for AC pipes: $k_N = 1{,}0.10^{-4}$mm, GI pipes: $k_N = 0{,}5.10^{-4}$mm	

$V = Q/A$ with Q is the flow in [m^3/s]
$A = [\pi/4] * \phi^2$ flow area in [m^2]

The functioning point of the pumps and the main should be geared to one another in order to make optimal use of them.
The total head H_t and the flow Q determine the functioning point. The diameter of the main depends on the allowable velocity and thus erosion in the main. Consequently the head loss due to friction H_f is an important parameter. To calculate a reasonable maximum capacity of a main, we accept a head loss not higher than 4 to 8 meters per kilometre of main. In distribution systems in most of the places no more head loss is available.
At places in the mains where air can be trapped, 'high points', air valves are needed. Air pockets in the main reduce the flow area and therefore the capacity. At low points wash-outs or scour valves should be placed in order to remove settled material at times.
In some cases there is danger for water hammer which asks for a special device to reduce the extreme pressures. Calculation of the exact pressures and devices is very complicated and mostly done with a computer programm. In this report only existing devices are checked if they are working properly.

Energy Supply
The energy supply to the system must be reliable. If not, a stand by power unit is required with a reasonable storage of fuel.

1.4 TREATMENT

1.4.1 Introduction

The kind of treatment used depends on the water quality of the source. A conventional surface water treatment, as used in Zambia, generally consists of the following components:
* floc forming and removal, by coagulation-flocculation-sedimentation
* filtration, rapid or slow
* disinfection

The capacity of the treatment should be large enough to satisfy the maximum daily demand at the end of the design period.

1.4.2 Floc forming and removal

General
Colloidal particles generally carry a negative electrical charge. Their diameter may range between 10^{-4} to 10^{-6}mm. They are surrounded by an electrical double layer (due to attachment of positively charged ions who form the ambient solution) and thus inhibit the close approach of each other. They remain finely divided, don't agglomerate and, due to their low specific gravity, they do not settle. A coagulant (generally positively charged Iron or Aluminium salts) causes compression of the double layer and thus the neutralization of the electrostatic surface potential of the particles.

The resulting destabilized particles stich together when contact is made. Rapid mixing (a few seconds) is important at this stage to obtain uniform dispersion of the chemical. Subsequent gentle and prolonged (several minutes) mixing, cements the still microscopic coagulated particles into larger flocs. These flocs then are able to aggregate with suspended polluting matter. When increased sufficiently in size and weight, the particles can settle to the bottom.

Coagulation

Generally used coagulants are aluminium and iron salts. The optimal chemical dosage will vary depending upon the nature of the raw water and its overall composition, as well as on factors as pH, temperature, the intensity and duration of mixing and the quantity of to be treated water. Aluminium and iron salts have considerable differences in their pH zone of good coagulation. For alum the pH zone for optimum coagulation is quite narrow, ranging from about 6.5 to 7.5. The comparable range for ferric sulphate is considerably broader, a pH range of about 5.5 to 9.0. The choice of coagulant mostly depends on availability and costs. Important is that they are always available.

For determining the optimum dosage of coagulant and pH, there is a relatively simple test, the Jar test. A Jar test has to be done regularly while the optimum dosage changes with changing raw water quality and the pH is influenced by the amount of dosed chemical.

A too low coagulant dosing causes insufficient coagulation while not enough particles are destabilized. Consequently floc will not grow very well and the removal ratio of the sedimentation will be low.

Overdosing of coagulant will form a turbidity source itself and flocculation and sedimentation will be less effective than with a optimal dosing.

To adjust the pH, if acid, lime can be dosed.

The dosing devices must give a continues, adjustable, flow rate and a constant concentration.

Very important is a proper rapid mixing to obtain a uniform dispersion of the coagulant. If not, the same effects as with a too low dosing will occur. Rapid mixing is done by creating high turbulence in the water, mechanical, by stirring, or hydraulic, with a stationary wave or baffle channel.

Flocculation

Flocculation is the process of gentle and continuous stirring of coagulated water for the purpose of forming flocs through the aggregation of minute particles. The efficiency of the flocculation process is largely determined by the number of collisions between the minute coagulated particles per unit of time. There are mechanical and hydraulic flocculators.

In mechanical flocculators the stirring of the water is achieved with devices such as paddles, paddle reels or rakes. A disadvantage is that they need a motor and thus energy.

In hydraulic flocculators, the flow of the water is influenced by small hydraulic structures, such as channels with baffles or flocculator chambers, to get a certain amount of turbulence. The disadvantage of hydraulic flocculators is that they are hard to clean and that no adjustment is possible to changes of raw water composition or to the water production rate of the treatment plant. Furthermore the head loss for local input of energy is often appreciable.

In the design of a flocculator both the velocity gradient (G) and the detention time (t) should be taken into account. The product G*t gives a measure for the number of particle collisions, and thus for the floc formation action.
The formula for computing the velocity gradient is:

$$G = \sqrt{\frac{P}{\mu * V}} \qquad \text{Velocity gradient (3.1)}$$

with:

G :	velocity gradient	[1/sec]
P :	power transmitted to the water, for hydraulic flocculators $P = \rho * g * \Delta H * Q$	[kilowatt]
V :	volume of water to which the power is applied; where applicable, the volume of the mixing tank or basin	[m^3]
μ :	kinematic viscosity of water	[m^2/sec]

For each individual flocculator, the optimal G.t value should be carefully selected, and taken as high as possible as is consistent with the optimal floc formation. High values provide many collisions, but again can cause disintegration of flocs.
Typical values are: G = 10 to 100 [1/s]
t = 1200 to 1800 [s]

Sedimentation
Sedimentation is the settling and removal of suspended particles. An important design criterion is the surface load s_0, which can be calculated with the following formula.

$$s_0 = \frac{Q}{A} \qquad \text{Surface load (3.2)}$$

s_0 :	surface load or settling velocity	[m/hr]
Q :	flow rate	[m^3/hr]
A :	surface area of sedimentation tank	[m^2]

In a settling tank three zones can be distinguished: an inlet zone, a settling zone and an outlet zone. Demands on these zones are to provide optimal settle conditions, without turbulence, short cutting or resuspension.

The inlet zone is to provide an equal distribution of inflowing water over the full cross-section of the tank. In the settling zone the flow must be stable and velocities not to high (in the range 4 to 36 m/hour). The outlet zone has to collect the cleared water evenly over the full width of the tank (a weir is commonly used for this in rectangular tanks).
There must be a regular sludge removal, by hand or with machinery. To much sludge will stimulate resuspension and consequently a lower settling efficiency in the tank.

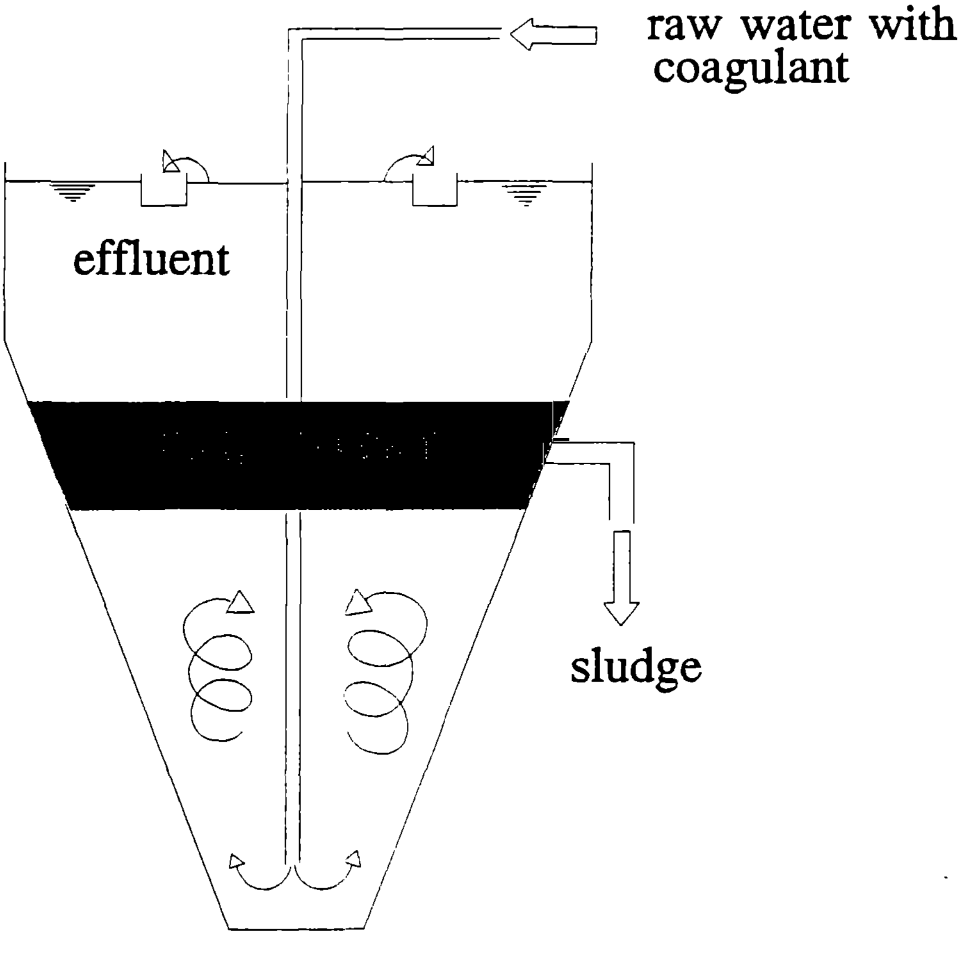
There are two types of settling: plain or discrete settling and flocculent settling. Plain settling occurs when raw water is not pre-treated by coagulation and flocculation. The surface load s_0 for plain sedimentation ranges from 0.1 to 1.0 [m/hr]. With plain sedimentation, only sand and larger suspended matter is removed. In settling tanks that receive pre-treated water there will be flocculent settling, which makes a surface load between 1 and 3 [m/hr] possible. In both cases, the lower the surface load the better the clarification.

Another distinction can be made between the different types of settling tanks used in Zambia.

a. Rectangular with horizontal flow
The efficiency is independent of the depth of the tank, however there are some restrictions. To keep the effects of turbulence, short-circuiting and bottom scour to a minimum, the tank should not be too shallow, at least 2 meters deep, and the ratio between length and width should be between 3 and 8. Within the restricted range for the surface load s_0 the horizontal velocity of flow, $v_0 = Q/BH$, will then be between 4 and 36 m/hr. A tank 2 metre deep could accommodate mechanical sludge removal equipment but small tanks are better cleaned manually.

b. Circular
Commonly used is the 'sludge blanket' type as pictured on the next page. Raw water with coagulants enters the tank at the bottom and flows up through the tank.
The inlet zone, were the diameter is small and were there is a lot of turbulence, serves as rapid mixing area. As the water flows up through the cone, turbulence and therefore the G-value decreases, and successively coagulation and flocculation take place. Under good operation, the last part of the cone holds the sludge blanket. The weight of the sludge blanket equals the upward force caused by flow resistance as the water flows through the sludge blanket. Up to some extent the sludge blanket is in balance and self regulating. As the blanket for some reason falls, the cone causes the blanket to compress which gives more resistance to flow and therefore a higher upward force. If the sludge blanket for some reason rises, the opposite takes place. But when for example the inflow rate is so high that the sludge blanket is pushed into the cylindrical part of the tank, the whole blanket will be washed out.
Under normal conditions the sludge blanket will function as a sort of filter and absorb the flocculated particles. The process will not work if the inflow rate or surface load is to low, due to lack of turbulence there will be no proper coagulation and flocculation. Normal values for the surface load s_0 range from 1 to 2 m/hr.

Upflow sedimentation tank



S_0 = 1 to 3 m/h

Sludge removal, by abstracting a part of the sludge blanket while the tank is in operation!, should be done regularly to prevent the sludge blanket to become too heavy and fall. A special pipe for this purpose is placed at the height were the sludge blanket should be under good operation.
When no coagulants are dosed, the sludge blanket will be absent and plain sedimentation will cause accumulation of more dense particles as sand grains in the cone. Efficient sludge removal then, can only be done by draining the tank completely(in many cases there is no drain valve so it must be done by pumping and removing the sludge manually). While the maximum allowable surface load for plain sedimentation (0.1 to 1.0) is lower than the one for coagulated water, the inflow rate must be lowered when no coagulants are dosed.

1.4.3 Filtration

Filtration is often the final treatment step.
Important are the characteristics of the filter media, mostly sand. These can be determined by making a curve of the grain size distribution of the sand. Two parameters are: the effective size, d_{10}, and the coefficient of uniformity, $U = d_{60}/d_{10}$.
Another important parameter is the filter rate, $v = Q/A$.

Filter control
In sand filters negative pressures, water pressures below atmospheric, must be avoided. Gas bubbles would form and accumulate in the filter bed, increasing the resistance against the filtration flow. Gas bubbles of large size may even break up the filter bed and create fissures through which the water would pass downward with insufficient purification. The maximum allowable head loss over the filter bed is thus limited to the depth of the supernatant water plus the resistance of the clean filter bed at the minimum filter rate. An overflow weir in the effluent line could prevent negative pressures completely. The difference in level between the weir and the top of the filter bed(weir placed lower) should not be more than the head loss due to resistance in the clean filter bed plus the resistance in the effluent piping, again for the minimum filter rate. To avoid all risks, the weir could be placed at the same level as the top of the filter bed. In the last case, also the filters can not run dry anymore.

Basically there are two types of filter control, inlet controlled and outlet controlled filters.
Inlet controlled filters, pictured on the left page. The effluent flow of these filters are not influenced with the outlet device which for example consists only of an overflow weir. The only control during operation is at the inlet of the filter. If the influent flow is constant in time, there will be a rising supernatant level as the filter bed clogs during operation. If the supernatant level is kept the same, by adjusting the inflow, it is called declining rate filtration, while the filtration rate decreases as the filter clogs.
Outlet controlled filters. The supernatant level and the filtration rate can be influenced with the outlet device, mostly a valve or adjustable overflow weir. Filtration can be maintained at the same rate, by frequently decreasing the resistance over the outlet device as, due to

clogging, the resistance over the filter bed increases. Important is the minimum resistance over the outlet device, which should be large enough to prevent negative pressures to occur.

Slow sand filtration (SSF)

General

The main purpose of slow sand filtration is the removal of pathogenic organisms from the raw water, in particular the bacteria and viruses responsible for the spreading of water related diseases. Filtrate, direct from a good working slow sand filter, is drinkable without any risks.
Slow sand filters are also very effective in removing suspended matter from the influent water. However, the clogging of the filter bed may be too rapid, necessitating frequent cleanings. Slow sand filters are able to handle raw water with a turbidity lower than 50 NTU which is not exceeded for more than a few days. Best clarification occurs with raw water turbidities lower than 10 NTU.
Filter rates are low, between 0.1 and 0.3 m/hr, so fairly large areas are required. These low rates are necessary for the removal of organic matter.
In slow sand filters suspended matter is caught by a very thin top layer, the schmutzdecke, while the water percolates through the whole filter bed. Under the top layer there is layer with bacterial activity, which extends over a depth of about 0.6 metre. In this layer the actual break up of organic matter such as viruses and bacteria takes place. It takes about two month to build up an effective biological layer. To prevent organic matter to pass the filter, the effective bed thickness should never be less than 0.7 m. The initial bed thickness should be 0.3 to 0.5 m more, to allow for a number of filter scrapings before resanding is necessary.

Sand

The effective size d_{10} of the filter sand should be between 0.1 and 0.3 mm. Too small sand grains cause a lot of resistance against flow, and therefore a very low filter rate and short filter runs. Larger grains will give higher filter rates but also a decreasing water quality.
The Uniformity coefficient of the sand should rather not be higher than 3.

Cleaning

When the maximum allowable head loss is used by resistance over the filter bed, due to clogging, the filter should be cleaned. This must be done by lowering the water level in the filter to about 0.2 metres below the surface of the sand bed and scraping of a top layer, about 2 to 3 cm thick, of dirty sand. When the cleaning has been completed the filter is slowly refilled with filtered water from underneath.
Scraping off only a too thin layer causes a choked up top layer and consequently lower filter rates and shorter filter runs.

Rapid filtration

General

The main purpose of rapid filtration, when surface water is treated, is lowering the turbidity. The effluent can still contain pathogenic bacteria and viruses. Courser sand than with slow sand filtration is used and average filtration rates are much higher, 5 to 10 m/hr. Consequently, impurities contained in the raw water penetrate deep into the filter bed, thus asking for another cleaning method which is back washing.
The rapid filters discussed here are single media rapid sand filters.

Sand

The effective size d_{10}, for single media rapid filters, as often used in Zambia, should be between 0.4 and 1.2 mm, the coefficient of uniformity should be below 1.5. Again, with large grains the filtration rate can will high but the effluent quality decreases. Also high back-wash rates are required when large grains are used in order to obtain a 20 percent expansion of the bed. For small sand grains the filtration rate will be low and it is difficult to create enough scouring force during back-wash(grains smaller than 0.8 mm).
The coefficient for uniformity is very important now. When sand which is not very uniform will be back-washed, smaller grains will come on top of the bed and larger grains more down in the bed. Impurities will than penetrate less deep into the bed, shortening the filter run and causing cake filtration and forming of mud balls.
A granular underdrainage should prevent sand to be washed out with the effluent and is also necessary for an equal distribution of back-wash water.

Control

A lot of rapid filters work with a rising supernatant level to compensate the rising filter resistance due to clogging. Important is that the supernatant level is at its maximum or back-wash level before impurities break through the filter bed. Otherwise the effluent quality would decrease at the end of a filter run.

Backwash

By directing a flow of water and possible air upward through the filter bed for a few minutes, the bed is expanded and dirt is removed with the back-wash water. The velocity of the upward water flow should be high enough to produce an expansion of about 20 percent and enough scour to loosen all clogged material.
For a filter bed of sand (specific weight: 2.65 g/cm^3) typical backwash rates giving about 20 percent expansion are listed in table 1.

Typical	Backwash rates [m^3/m^2/hour]								
t [°C] of wash water	d [mm], average grain size of filter sand 0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2
10	12	17	22	28	34	40	47	54	62
20	14	20	26	33	40	48	56	64	73
30	16	23	30	38	47	56	65	75	86

Table 1: typical backwash rates

If back washing is done by directly pumping the water into the filter bed, fairly large, and expensive, pumps are required. An interesting alternative is using a wash water reservoir. Relatively small pumps can fill the reservoir in between cleanings while the reservoir can emptied within a few minutes during backwash. The reservoir generally should have a capacity between 3 and 6 m^3 per m^2 of filter bed area and it should be placed 4 to 6 metres above the water level in the filter depending on the head loss.
For sand with a grain size smaller than about 0.8 mm, the scouring force of the rising water may be inadequate to keep the filter grains clean in the long run. Also with too low backwash rates filter sand can not be kept clean enough. This may cause problems such as mud balls and filter cracks and therefore deterioration of the filter operation and effluent quality. These can be prevented by providing an additional scour through air wash. Filter cleaning now starts back washing with air for a few minutes at a 30 to 50 m/hour rate, usually with a water wash at 10 to 15 m/hour rate followed by a back wash with only water at normal back wash rates.
An equal distribution of wash water and air over the filter bed is necessary, for which a good under drainage of different grain sizes is indispensable.

1.4.4 Disinfection

Disinfection is necessary for destruction or complete inactivation of harmful micro-organisms present in the water. Even when water is filtered by a good working slow sand filter disinfection is necessary because water can become contaminated during distribution.
Disinfection is influenced by the following factors:
* The quality and nature of the water and the organisms it contains.
* Type and concentration of disinfectant.
* Temperature and pH of the water
* The time of contact between water and disinfectant
* Mixing

Clean water such as effluent of a slow sand filter, needs less disinfectant than water that contains more micro-organisms.

As general rule for chlorination, in order to be on the safe side, one can demand that water at every tap must at least contain a trace of free, residual, chlorine, which can be tested relatively simple with a residual chlorine comparator. Continues dosing with a constant concentration of chlorine and proper mixing are very important for an equal distribution of the disinfectant over the treated water. For example dosing in a hydraulic jump provides good mixing conditions. A contact time of at least half an hour is necessary.
When surface water is used, a dosage of about 2 to 3 mg/l or p.p.m.(parts per million) of active chlorine is than required.
For disinfection High Test Hypochlorite (HTH) is generally used in Zambia. This powder contains 70 percent active chlorine.

Chemical storage
Coagulants, disinfectants and other chemicals need to be stored properly, kept away from anyone not involved. Availability requires a storage of enough chemicals for the time no new is coming in.

1.5 CLEAR WATER TANK (CWT)

De required capacity of a clear water tank depends on its function. When it is only meant as buffer between treatment and clear water pumps as one of them fails it can be fairly small, for about one hour of production, just to have time to stop necessary pumps. This means a capacity of about 5 to 10 percent of the daily production, which is also large enough to provide enough contact time for the disinfectant. If back wash water is directly taken from the clear water tank, this must be accounted for in the capacity of the tank.
When the capacities of treatment(with the raw water pumps) and clear water pumps are not equal, the clear water tank should be able store the amount of treated water over periods the clear water pumps are not working.

1.6 SERVICE RESERVOIRS

The total capacity of service reservoirs should be large enough to accommodate the cumulative differences between water supply and demand. This means a capacity of about 28 percent of the peak day demand or 34 percent of the average day demand (peak day = 1.2 * average day demand). When there is only a 12 hour supply the required capacity is about 26 percent of the average day demand. For fire fighting 60 m^3/h during two hours is needed, 120 m^3 in total. To secure the supply during short breakdowns a reserve is required, larger reservoirs increase the guaranty for supply.

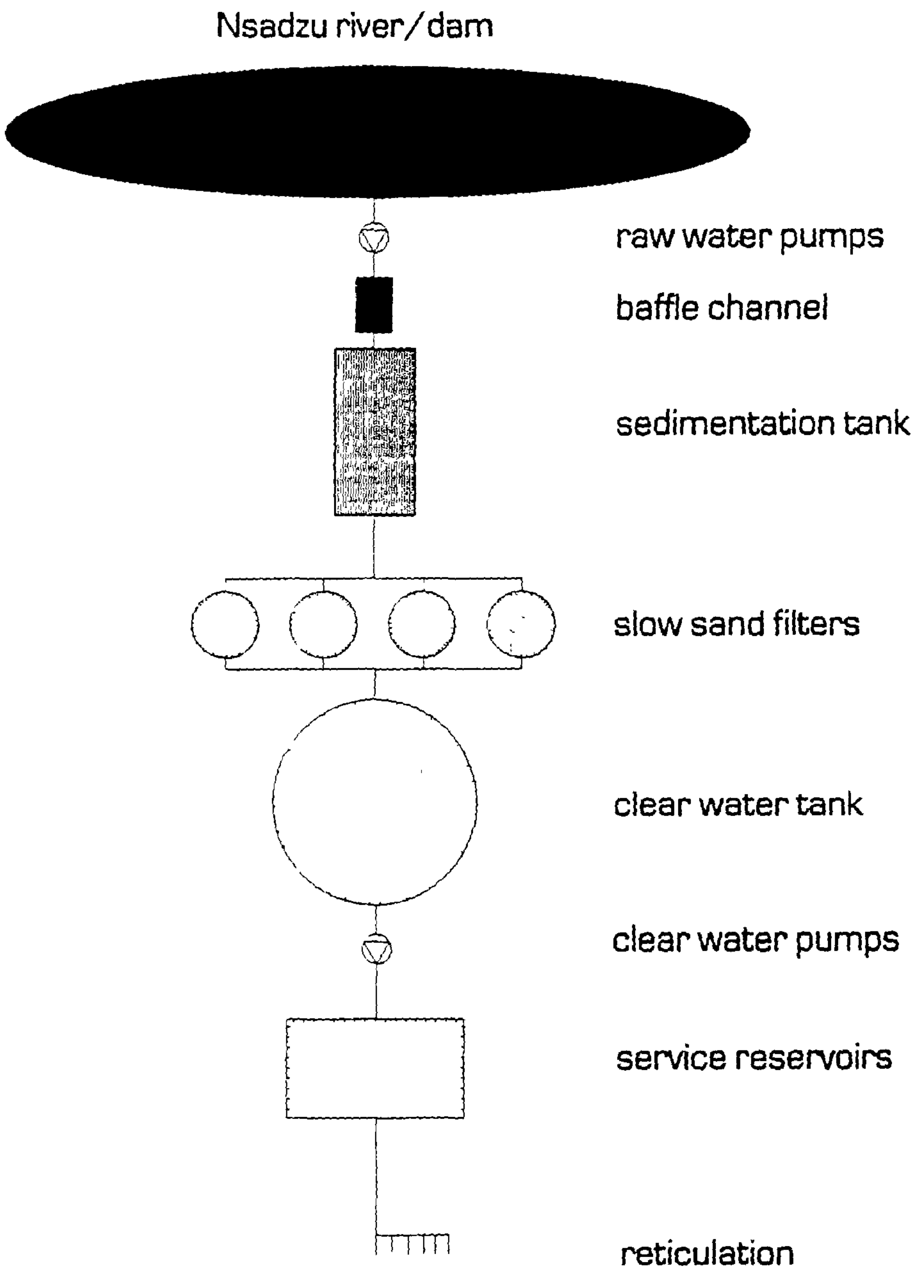
1.7 DISTRIBUTION

Distribution mains should be designed at the peak hour demand which is assumed to be 1.8 times the average hour on a peak day or 2.2 times the average demand on an average day. Very important are maps with pipe sizes and were they can be found, combined with contour lines. It is than possible to determine available head losses and it is always possible to check everything in the field on the map.
A first check on the capacity of the system can be done by determining the capacity of the pipes that come out of the reservoirs and feed the distribution. Calculation is done by allowing a head loss of 4 to 8 m/km, depending on the system and the available head loss. Best is a 24 hours a day supply to the town and enough pressure in all the pipes and at all the taps. If not, the water quality will decrease as water can stay too long within the system and ground water can penetrate into pipes with a too low pressure. Also air bubbles can block pipes so regular deaeration is necessary if the supply is not 24 hours a day.
In order to save water it is possible to stop supplying during nights, while than most of the water consumption is through leakages, but the above mentioned disadvantages must be kept in mind.
Overall performance of the system should be checked regularly. Leakages, blockages and taps that get no water should be detected and repaired. Especially leakages can spill huge amounts of water.
The tap water quality should be monitored. The turbidity should be very low, pathogenic organisms absent and there should be no colour or odour. Also the water must be chemically stable. Turbidity can be checked with a turbidity meter, pathogenic organisms are in most cases absent or inactive if there still is disinfectant in the water, colour and odour can be checked on sight and smell and simple chemical tests are pH and alkalinity.

1.8 SYSTEM CAPACITY

The capacity of the water supply system should equal the demand at the end of the design period. The peak day demand then is about 1.2 times the average day demand and the peak hour demand is about 1.8 times the average hour on a peak day and thus equal 2.2 times the average demand per hour. The capacity of the water source, the raw water main and the treatment plant should be designed for the peak day demand, and the distribution system for the peak hour demand.
To prevent wastage of water, irregular operation and the need of large reservoirs, the capacity of all the components should be geared to one another.
For the water demand it is necessary to account for wastage and leakages while otherwise the consumption can be much higher than calculated. Also the percentage of the population that is connected to the system is of importance. People that are not connected will also drink water and wash their clothes somewhere and in many cases will use water from the water supply system for this. Especially with public stand posts the illegal draw off can be large.

Chadiza



system layout

2 SYSTEM ANALYSES

2.1 CHADIZA

Introduction

The water sypply scheme of Chadiza embodies an intake with a reservoir at the Nsadzu river, the treatment works consist of a baffle channel, two regtangular horizontal flow sedimentation tanks and four slow sand filters. Disinfection takes place at the clear water tank from where the water is pumped to two high ground reservoirs and one elevated reservoir. There the distribution to town takes place by gravity.

Intake of raw water

While both the capacity of the reservoir at the moment (i.a.due to silting) and the base flow of the river are not known, it is not possible to calculate a maximum abstraction of raw water in the dry season.
Erosion of cultivated land is said to be a problem in the Eastern Province. This causes high turbit rivers and silting in the reservoirs which reduces the capacity and gives high suspended solid loads on the treatment.
The spillway of the reservoir dam is broken, so all the energy of the overflowing water is dissipated just in front of the foot of the dam. Consequently the protected foot has deteriorated badly and undermining of the structure is beginning to occur, which will results in more seepage and eventually imbalance of the dam.
The raw water pumps can not be controlled from the treatment site where the operators stay, and only one of the two pumps, the 62 m^3/h pump, starts itself when the power supply is restored after a power failure. This pump is used at night.
The maximum capacity of the pumps is 91 m^3/h which is more than the gross 100 % connection demand.
It takes a lot of time to stop the raw water pumps, and while there is a regular overflow in the sedimentation tank, a lot of water and energy is wasted.
All pumps are electrical pumps and the power comes from the Katete Hydro Power Plant, with every week about two breakdowns.

Treatment

Coagulation
In the rainy season the alum dosing is continuously, at a rate of 10 kg a day, with an apparatus built by the operator in charge himself. The apparatus works without mayor problems.

The amount of alum dosed is constant, no Jar-tests are carried out and variations in the inflow of the treatment and the turbidity are not taken into account, thus the alum concentration and the efficiency of the coagulation/flocculation/sedimentation processes are constantly varying.
No visual control of the coagulation/flocculation process is carried out.
The rapid mixing, in a small sloping baffle channel just after the alum dosing, is sufficient, so when the right amount of alum is dosed good coagulation is possible.

Flocculation
After the baffle channel there is only a flow dividing chamber
but no flocculation chamber. Thus the process is reduced to flocculent settling only with too small floc sizes and consequently a low efficiency of the sedimentation tank.
The G_t-value is almost zero because no power is added.

Sedimentation
Since there is almost no flocculation, the sedimentation can be seen as plain sedimentation: With a surface area of 72 m^2 the average production of 42 m^3/h gives a surface load of S_0 = 0,58 m/h for plain sedimentation, which is reasonable. With flocculent settling the maximum capacity is 216 m^3/h.
The inlet zone, with an equal distrubution of water over the cross section of the tank is alright. The length/width ratio for the settling zone is 4, which is also ok. The outlet of the tank is not good while it is at one point and not over the whole width of the tank. This causes short circuiting.
With a raw water turbidity of 65.7 NTU and after sedimentation 42.4 NTU the removal ratio is only 35 % which emphasizes that the floc forming and removal is not properly working. With proper flocculation a maximum surface load of 3 m/h is possible.
The depth varies from 2.8 to 4 m. This is sufficient since this is more than 2 meter. The ratio between length and with is 4, this is OK.

Filtration

Control
The filters are designed as inlet controlled. During use, the supernatant level should rise as the filter bed clogs.
At the time of visit supernatant levels were constantly at their maximum level. Consequently declining rate filtration occurred.
The malfunctioning of the pre treatment causes high turbidity loads on the slow sand filters. During the visit, in the dry season, the turbidity was already 42 NTU and for the rainy season much higher turbidities are expected which will shorten filter rates and filter runs.
The average filter rate of 0.12 m/h is quite low. The maximum average filter rate of 0.25 m/h over four filters, gives a total rate of 4 * 0.25 * 176 = 176 m^3/h.
The thickness of the filter bed is approximately 1 metre which is sufficient.
The sand parameters, effective size d_{10} = 0.4 mm and uniformity U = 4.1, are not within the range given for slow sand filter sand. In this case the effective size of the grains is too

large and there is not enough uniformity. In this way short circuiting and low effluent qualities can occur.

Only two filters are used at the same time. The third one is cleaned or stand by and number four has no sand. Before cleaning the filters are dried completely, which destroys the biological layer in the filter bed. As a result the biological treatment will never be optimal because it takes about two month to built up a new biological layer. This means too much organic matter such as pathogenic organisms and acids in the soil, which give the water a clear yellow brownish colour during the rainy season, will pass the filter bed.

Disinfection
The chlorination device is broken, so continues chlorination of the filtered water is not possible. Instead, the daily amount of HTH-powder is dosed at once in the morning. This gives an initial concentration active chlorine $[Cl^-]$ = 0.8 p.p.m. in the morning when the CWT is half full and pumping to the township is started, dropping to nearly zero at the end of the day (in the clear water tank!). This means part of the day not chlorinated water is distributed. Even the initial concentration in the morning is too low, because 2 to 3 p.p.m. is required.
Another point is the dosing place itself. Dosing is done straight into the clear water tank, were mixing conditions are very poor.
The contact time is large enough. With the tank half full the average contact time in the clear water tank is about 4 to 5 hours.

Clear water tank

The tank has a capacity of 454 m^3, which is 45 % of the average daily production. This is enough at the moment. It is also 28 % of the peak day production with the gross 100% demand which is sufficient.
The capacity of the tank is large enough as buffer for the filters and the clear water pumps as well as for the retention time needed after chlorination.

Clear water pumps

For each of the two supply areas (township and hospital) there is only one working pump, what makes the supply very vulnerable.

Clear water main

The size of the clear water main is 225 mm AC. When $k = 1.0 * 10^{-4}$ and $\Delta H = 4$ m/km than Q = 150 m^3/h.
The main has six air valves of which four are badly leaking. Two have been completely blocked by the plumber because there are no spare parts available to repair or replace the airvalves. This is why there is danger of air pockets, thus reducing the discharge of the main.

Service reservoirs

The total capacity of the reservoirs is 958 m^3. This is 96 % of the daily production and 60 % of the peak day production with the gross 100% demand which is sufficient to store the differences in water production and demand over the day and as a small reserve for supply during breakdowns.
One of the reservoirs is badly leaking, much water is wasted here.

Distribution

Leakages of pipes and taps often occur. Lack of spare parts make repairs difficult.
The water supply is rationed during the day and during nights there is no supply at all. The worst neighbourhood only gets water for three or four hours a day. This causes probably air blockages and a decreasing water quality.

CHADIZA CAPACITY SHEET		
Production: 42 m^3/h	Gross Demand now: 46 m^3/h	Gross 100 % demand average 55 m^3/h peak day: 66 m^3/h peak hour: 119 m^3/h
maximum	capacity in m^3/h	100% parameters
raw water pumps	91	73 % of the day
raw water main	> 91	dH = ? m/km
sedimentation	72 (plain) 216 (flocculent)	S_0 = 0.92 m/h
filters	176	q = 0.10 m/h
clear water pumps	91 + 30	73 % of the day
clear water main	150	dH =0.78 m/km
distribution	?	dH = ? m/km

Capacities

The raw water pumps are pumping approx. 1.750 m^3/day, (if the pumping hours are taken as the operators say), into the treatment while the total production of the system is only 1.000 m^3/day. Wastage of energy and water, which is mainly overflowing the sedimentation tank, is then approx. 750 m^3/day at the treatment only, which is 75 % of the daily production. Due to power failures and interference of the operators when there is a overflow, the total wastage will be less, but still considerable.
The raw water and clear water pumps have the same capacity, but the filters, as they were used during the visit, can not handle the total flow and are more or less the bottle neck of the system.

Conclusions

While the effective storage of the intake reservoir is not known, it is impossible to determine the maximum monthly withdrawal for the dry season. To much abstraction can result in shortages at the end of the dry season.
A lot of water and energy is wasted by overflowing as the capacity of one of the raw water pumps is too large compared with the present capacity of the treatment. Also pumping is with intervals and with different pumps, causing an irregular load on the treatment.
The dosing device for alum is build by one of the operators himself and is working very well. But as there is no proper flocculation the sedimentation process is limited to plain settling with a low efficiency. As a result the filters have to handle high turbidity loads, especially during the rainy season, which shortens the filter rates and filter runs.
The filter sand does not meet the specifications given for slow sand filter sand.
The filters can not handle the total inflow of the raw water pumps, which causes declining rate filtration with a constant (high) supernatant level instead of constant rate filtration with a rising supernatant level.
The cleaning procedure causes deterioration of the biological layer while with every cleaning the filter bed is dried completely. Consequently the biological clarification is not very good.
Disinfection is not optimal due to a broken chlorination device and insufficient mixing. Free chlorine concentrations are varying between 0 and 0.8 p.p.m.
All components are able to handle the gross 100% demand, only while there is almost no flocculation the sedimentation will not be very effective and the filters will be the bottle neck of the system as they clog too fast with high turbidity loads.

Recommendations

Repair the spillway of the reservoir dam and determine the present effective storage of the reservoir in order to be able to calculate the maximum monthly withdrawal.
Use only the smallest raw water pump, 62 m^3/h, to pump water to the treatment, the larger one can function as stand by for emergencies. In this way there is a constant flow through the treatment which can be handled by all the components and the total production than equals the gross 100% demand.

In order to get proper flocculation, a new flocculation unit needs to be added to the existing system. As an alternative the possibility of roughing filtration as only pre treatment can be investigated. The present sedimentation unit could serve as roughing filter tank.

While the present sand specifications are not very good, it is best to renew or sieve the sand. The d_{10} should then be between 0.1 and 0.3 mm and U below 3.
To improve the filter control 4 filters should be used at the same time and the influent flow (with only the small raw water pump working) must be equally divided over these 4 filters. This will result in constant rate filtration with a rising supernatant level. A filter has to be cleaned as the supernatant level has risen to its maximum level.
The biological layer can be kept intact by not drying the filter bed completely during cleanings, but just draining it to about 0.2 metres under the surface of the bed as is necessary for manual cleaning.
The cleaning procedure than changes to the following:

a. drain the filter to approx. 0.2 metres beneath the sand surface
b. directly after this scrape off the upper 2 to 3 cm of the sand bed
c. refill the filter again by backfilling.

For good disinfection it is necessary to repair the chlorination device and start dosing continuously 3 to 4 grams of HTH powder per m^3 treated water. The dosing place should provide good mixing conditions, for example at the place where filtered water flows into the clear water tank.

To prevent air blockages in the clear water main air valves should be repaired.

2.2 CHIPATA

Introduction

The water supply of Chipata is build up by two production units, Lutembwe 1 and 2, each with a raw water reservoir and treatment plant and each with more or less the same capacity. The two production units are separately discussed here. The water demand figures to compare the production capacities with are split equally over the two units, giving each unit 50 % of the total gross demand to produce.
All pumps are electrical pumps and power is normally certain for 24 hours a day.

LUTEMBWE 1

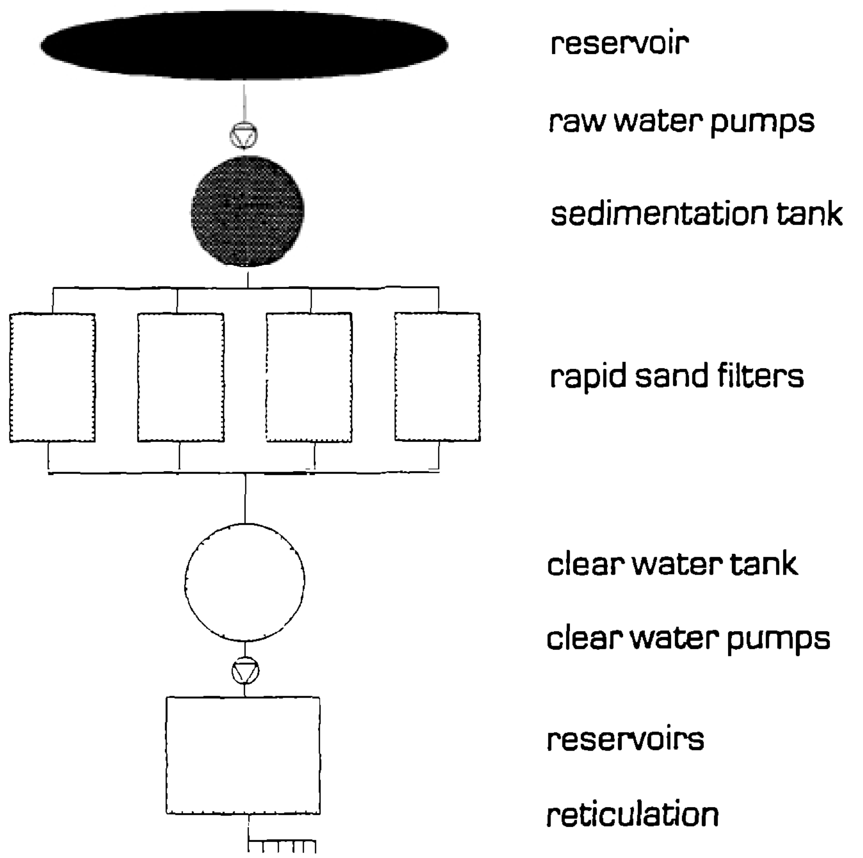
Introduction

The water sypply scheme of Lutembwe 1 embodies an intake with a reservoir, the treatment works consist of a circular, combined floc forming/sedimentation tank, accelator type, and four rapid filters. Disinfection takes place at the clear water tank from where the water is pumped to the high ground reservoirs. From there the distribution to town takes place by gravity.

Intake of raw water

There is a reservoir with a design capacity of 1.5 million m^3. Silting, due to erosion, probably reduced the capacity of the reservoir. A very indication for the maximum withdrawal, with a safety factor of 1.5 and 7 month long dry season is 140.000 m^3/month or 200 m^3/h. This is just enough for the average day demand when the production equals the gross 100% demand.
Since raw water is abstracted from a large reservoir, the quality of the water is good and fluctuations in quality are confined.
The intake tower is situated far from the shore and at several levels water can be taken in, which is important for as well water quality as water quantity.
There are two raw water pumps, one is a spare pump.
The capacity of both pumps is approximately 200 m^3/h.

Chipata
lutembwe 1



system layout

Treatment

Floc forming and removal
Alum is added by an alum feeder with a grinder. The dosage of alum is not very precise. Twice a week a Jar-test is done and depending on the result of this test, the dosage of alum is adjusted.
floc forming and removal takes place in one tank, an 'accelator' type. Raw water is brought in in the inner circle were flocculation takes place under mechanically stirring as the water flows down. Then, down in the tank, the water flows out of the inner circle to start flowing up again through the outer circle and sediments can settle in the cone under the tank.
When no alum is dosed, plain sedimentation gives a maximum surface load of 1 m/h and thus a maximum production of 69 m^3/h.
When alum is dosed, the maximum production is 207 m^3/h.
The surface load at time of visit $s_0 = 2.6$, alum was dosed so this is ok.
A sludge scraper removes the sludge in the sedimentation tank. At time of visit the device was working ok.
At time of visit the raw water had a turbidity of 14.2 NTU and after sedimentation 7.89 NTU which gives a removal ratio of 45%. The ratio does not seem very high, but that is always with low turbidities.

Filtration

Control
The filters are outlet controlled. Depending on the resistance over the filter bed, the outlet valve can be adjusted.
This is supposed to be done automatic by a floater on the supernatant level, connected to the outlet valve. Since this device broke down, the valve is controlled by hand. This is done ok but the disadvantage is that faults might occur.
At time of visit the turbidity of the water after sedimentation was 7.89 NTU, after filtration the turbidity was reduced to 2.72 NTU, giving a removal ratio of 65% which for low turbidities is good.
The maximum filter capacity, when the maximum filter rate is 10 m/h, is 400 m^3/h.

Sand
The filter bed consists of 80 cm of sand, this is enough. The d_{10} of the sand was found to be 0.85 mm, which is good. The uniformity U was 1.8 which is too high for a rapid sand filtration.

Backwash
Backwashing is done three times a week , first 30 minutes with air, than 15 minutes with both air and water. Since the d_{50} is approximately 1.4 and the filter area is 10.08 m^2, a back wash rate (when only water is used) of 887 m^3/h is needed in order to have enough

scour to keep the sand grains clean. This rate can be a little lower when air is used, but it needs to be high. The pumps, that are used now, have not such a high capacity. This why the present backwash procedure takes so much time and water in order to clean the sand thoroughly. Building backwash reservoirs might be a good alternative, using less time and water with the same results.

Disinfection

Every two hours 200 g HTH powder is dosed in the filter effluent line to the clear water tank. This is 1.8 kg per day, so the average free chlorine concentration is 0.38 p.p.m. which is not enough. Mixing conditions are reasonable while in the effluent line and at the end where the water enters the clear water tank. The retention time is long enough before the water is distributed.

Clear water pumps

The capacity of the clear water pumps is 183 m^3/h. There is one stand by pump.

Clear water main

The diameter of this main is 300 mm AC. When k = 1 * 10^{-4}, and ΔH is 4 m/km, than the maximum capacity is 316 m^3/h.

CHIPATA CAPACITY Lutembwe 1	SHEET 50% of Chipata's	demand
Production: 182 m^3/h	Gross demand now: 150 m^3/h	Gross 100 % demand average: 200 m^3/h peak day: 240 m^3/h peak hour: 432 m^3/h
maximum	capacity in m^3/h	100% parameters
raw water pumps	200	120% of the day
raw water main	?	dH = ? m/km
sedimentation	69 (plain) 207 (flocculent)	S_0 = 3.5 m/h
filters	400	q = 6.0 m/h
clear water pumps	183	130% of the day
clear water main	316	dH = 2.3 m/km

LUTEMBWE 2

Introduction

The water sypply scheme of Lutembwe 2 embodies an intake with a reservoir, the treatment works consist of a regtangular, horizontal flow, sedimentation tank and two rapid filters. Disinfection takes place at the clear water tank from where the water is pumped to the high ground reservoirs. From there the distribution to town takes place by gravity.

Intake of raw water

There is a reservoir with a design capacity of 6 million m^3. Silt in the reservoir reduced this capacity. The real capacity now is not known. When the maximum acceptable draw off is needed, the real capacity should be measured. A rough indication, with a safety factor of 1.5 and a dry season of 7 month is 570.000 m^3/month or 790 m^3/h. This is more than enough for even Chipata's total gross 100% demand.
Since raw water is drawn of from the lake, the quality of the water is reasonable good and the fluctuations in quality are confined.
There are two raw water pumps, one is spare.

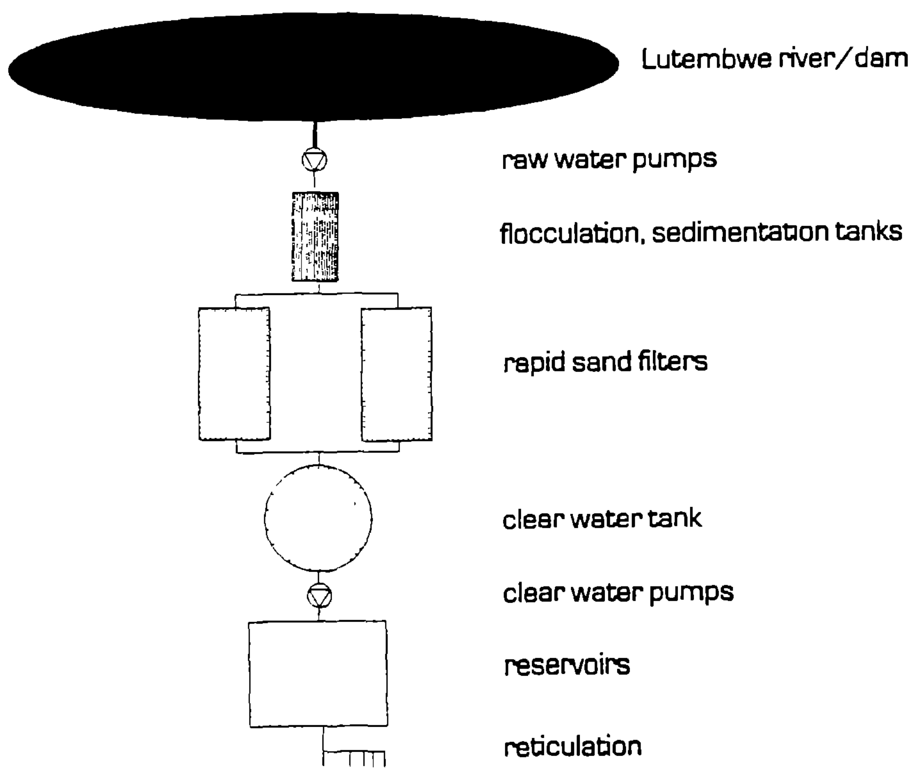
TREATMENT

Floc forming and removal

Coagulation
Rapid mixing is done mechanical.
Alum is dosed only in the rainy season. This means that in the dry season the only pre treatment is plain sedimentation. It depends on the turbidity of the raw water, whether this is right or not.
The amount of alum dosed depends on the result of a Jar test, which is done twice a week.

Flocculation
The flocculation tank is a vertical flow baffled chamber flocculation with decreasing G-value which seem sufficient.

Chipata
Lutembwe 2



system layout

Sedimentation

The two sedimentation tanks have an area of approximately 40 m^2 each.
The ratio Length/Width is 2, which is not enough, giving danger of disturbed or unstable flow with consequences as short circuiting and eddies.
S_0 = 2.38 m/h now, this is good as long as alum is dosed. At time of visit, no alum was dosed so the surface load was too high as for plain sedimentation s_0 must be between 0.1 and 1 m/h. This gives more or less the same results as just by passing the sedimentation and is only to justify if the turbidity during the whole dry season is low enough for direct filtration.

Filtration

Control

The two rapid filters have an area of 18 m^2 each.
The filters are outlet controlled filters. The outlet valves are controlled by hand. The filter rate is measured and can be read on a meter.The outlet valves have to be opened in such a way that the filter rate stays constant. This is done well.
The average filter rate at time of visit was 5.3 m/h. The maximum filter capacity, when the maximum average filter rate is 10 m/h, is 360 m^3/h.

Sand

The d_{10} of the sand was found to be 0.85 mm, which is good. The uniformity U was 1.8 which is too high for a rapid sand filtration and can cause grading of the sand grains during backwash.

Backwash

The backwash tank has a capacity of 100 m^3 so 5.6 m^3/m^2 is available. This is enough.
The backwash tank is situated approximately 8 m above the supernatant level of the filters which is sufficient.

Disinfection

Every two hours 200 g of HTH powder is dosed at once in the clear water tank. This gives an average of 0.74 p.p.m. which is not enough. Mixing condition are not very good causing a varying concentration of free chlorine in the distributed water. The contact time is ok, approximately one hour.

Clear water tank

The tank has a capacity of 450 m^3, this is 10 % of the average daily production and 8 % of the peak day for the gross 100% demand. While the service reservoirs function as buffer for the varying demand from town, this 10 % must be enough.

Clear water pumps

There are three pumps, Q = 190 m^3/h each. There are two stand by pumps which guaranty continues operation.

Clear water main

The diameter of this main is 300 mm AC. When k = 1 * 10^{-4}, and ΔH is 4 m/km, than the maximum capacity is 316 m^3/h.

CHIPATA CAPACITY Lutembwe 2	SHEET 50% of Chipata's	demand
Production: 190 m^3/h	Gross demand now: 160 m^3/h	Gross 100 % demand average: 200 m^3/h peak day: 240 m^3/h peak hour: 432 m^3/h
maximum	capacity in m^3/h	100% parameters
raw water pumps	208	115% of the day
raw water main	?	dH = ? m/km
sedimentation	80 (plain) 240 (flocculent)	s_0 = 3.0 m/h
filters	360	q = 6.7 m/h
clear water pumps	190	126% of the day
clear water main	316	dH = 2.3 m/km

Distribution

The two big service mains are 300 mm each, from both plants to the service reservoirs. When $k = 1 * 10^{-4}$ and $\Delta H = 4$ m/km, then $Q = 316\ m^3/h$ for each main.
The connection percentage of almost 80 is very good. Leakages and blockages are repaired quick and there is a program to determine total losses through leakages. Still there are some problem areas, compounds on higher grounds where water is available for only a few hours a day and people fall back on (polluted?) shallow wells.

CHIPATA CAPACITY SHEET Total demand		
Production: 347 m^3/h	Demand now: 312 m^3/h	100 % Demand average 400 m^3/h peak day: 480 m^3/h peak hour: 864 m^3/h
maximum	capacity in m^3/h	100% parameters
distribution	?	dH = ? m/km

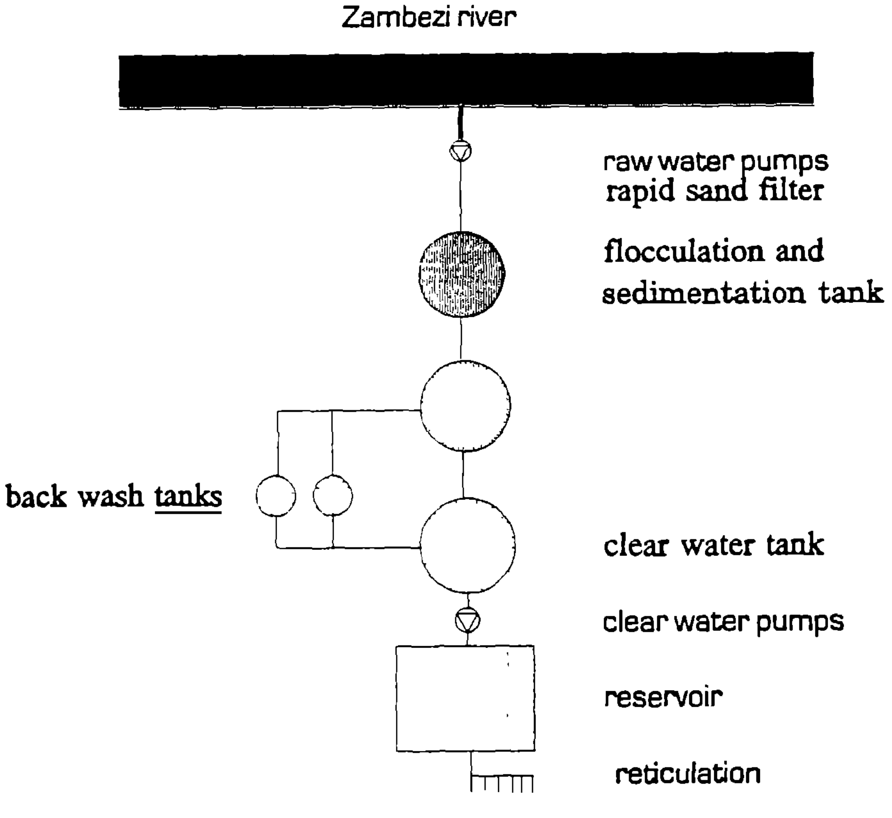
Conclusions

The Chipata water supply system looks very good. Most of it is run very well and the design seems to work very well. The total production capacity approximates the gross 100% demand. Still there are some problems.
At Lutembwe 1 the backwash procedure takes a lot of time and uses a lot of water while the backwash rate is too low. Investigations could show if it is better to use a backwash reservoir or place larger backwash pumps, saving time and water.
At Lutembwe 2 no alum is dosed during the dry season, overloading the sedimentation unit which can only handle a s_0 of 1 m/h as plain sedimentation occurs. This is only justified if the turbidity of the raw water during all the time no alum is dosed is low enough for direct filtration.
At both Lutembwe 1 and 2 there no chlorination device or proper mixing, resulting in varying free chlorine concentrations in the treated water.

Recommendations

Investigation of alternative backwash procedures and facilities is necessary to show if it is possible to save time and water at Lutembwe 1.
A chlorination device is needed at both Lutembwe 1 and 2 and proper mixing conditions need to be created.

Chirundu



system layout

2.3 CHIRUNDU

Introduction

The water sypply scheme of Chirundu embodies an intake directly from the Zambezi river, the treatment works consist of a circular, combined, upflow floc forming/sedimentation tank that works with a sludge blanket, one rapid filter and two back wash tanks. Disinfection takes place at the clear water tank from where the water is pumped to an elevated reservoir. From there the distribution to town takes place by gravity.

Intake of raw water

Water is taken in directly from the Zambezi river. The basic flow of this river is enough, even in dry periods, to draw off water for Chirundu township.
Because of the low level of the Zambezi river at time of visit, the original intake was situated at a too high level. The pump was placed at a lower level next to the original structure to be able to take in water (less solid structure). Water is taken in through a tube with a strainer in front of the opening and just thrown into the river, which makes it very vulnerable.
Two pumps (one stand by), Q = approximately 13 m^3/h. The maximum capacity of the raw water pump is 9360 m^3/month. At the moment the gross demand is 9219 m^3/month which the pumps can handle. The gross 100 % connection demand is 15690 m^3/month which is too much for them.
All pumps are electrical pumps and power comes from the main network and supply is regular.
The head over the raw water main is not known. When a new pump has to be installed, one needs to know the head in order to get the right pump.
With $d = 75$ mm, $k = 1.5 * 10^{-4}$ and $\Delta H = 8.0$ m/km, Q is 11.3 m^3/h. At time of visit Q was 13 m^3/h so the capacity of the main is not sufficient, especially when the water production in the future increases.
There is a bulk meter, so the amount of water taken in is known.

Treatment

Floc forming and removal

Alum is dosed only in the rainy season when the turbidity of the water is high. This is ok when the turbidity in the dry season is low enough.
In the rainy season once a day 4 kg of alum is dosed in a small tank, next to the

sedimentation tank. Water is added during the day, so the solution gets diluted. The average dosage is 18 mg/l. When a higher turbidity of the water is noticed, the alum dosage is increased just on sight. Good coagulation depends on the right amount of alum. This amount should be determined by executing a jar test every day or week (depending on the variation of the turbidity). When operators in charge have a lot of experience, dosing on sight can be sufficient.

The sedimentation tank is of the circular 'sludge blanket' type as explained in the chapter 'theory' of this appendix. With surface loads between 1 and 3 m/h, Q between 16 and 48 m^3/h, the rapid mixing down in the tank and the flocculation in the cone are sufficient, but at low rates there is not enough energy brought in for proper coagulation and flocculation while the G-value also depends on the flow rate. Also with low flow rates the sludge blanket (if formed) will drop too low for regular removal of a part of the sludge blanket. At the plant operators expect the sludge to settle at the bottom where no drain can be found and at times empty the tank manually to remove the sludge.
When no alum is dosed the tank is used as a plain sedimentation tank: with a surface area A = 15.9 m^2 and an average production of 9 m^3/h, the surface load s_0 = 0.57 m/h, this is ok. For the surface load s_0 = 1, the maximum Q is found: Q= 15.9 m^3/h. During the rainy season the flow rate is too low for proper flocculent settling. For good coagulation and flocculation and the build up of a good sludge blanket the capacity must be between 16 and 48 m^3/h (s_0 between 1 and 3 m/h).

Filtration

Control
An overflow weir in the effluent line to the clear water tank protects the rapid filters against negative pressures. This is very useful for good operation. Filter control is done on sight: when the water level rises too high and the filters overflow, the back washing procedure is started.
The system is designed to be inlet controlled with a rising supernatant level and is treated like that, only backwashing should start before the filters overflow and water is wasted.
At time of visit the turbidity of the water after sedimentation was 1.75 NTU. This is quite low so no problems were expected with the filters.
The design filter rate is 5 m/h, at time of visit a rate of 2.2 m/h was found. A maximum average filter rate of 10 m/h, gives a maximum production of 60 m^3/h. The design rate gives a capacity of 30 m_3/h.
The tanks themselves are in bad condition, rust occurs everywhere and the construction is very old.

Sand
The uniformity of the sand, U = 2.34, is to high which causes grading during backwashing. The effective size of the sand grains calculated from a sand sample operators took appeared to be very small, d_{10} = 0.5 mm. The sand sample is probably

taken from the top of the sand bed where, due to the mentioned grading, the effective size will be smaller than down in the sand bed and the determined coefficient of uniformity will be lower than the average U over all the sand. Loads of mud balls and filter cracks in the sand bed emphasize that the presently used sand (and backwash procedure) is very bad. The resulting cake filtration decreases filter rates and mud balls cause short circuiting and a lower effluent qualities.

Backwash

Two backwash tanks of 9 m^3 each are built on top of the office, 3 to 4 m above the water level in the filter. For backwashing of a filter only one tank for one cleaning is used. While the filter bed area is 6 m^2 which requires a backwash reservoir with a capacity between 18 and 36 m^3 placed at least 4 m above the water level in the filter. The sand used, with d_{50} = 1.05, requires a backwash rate of 60 $m^3/m^2/h$ (T = 20°C), or 360 m^3/h for the whole filter, in order to get enough expansion and scour to keep the sand bed clean. The two backwash reservoirs thus need to be emptied each within 90 seconds making the total backwash time 180 seconds.
The present situation is very bad. Mud balls have accumulated in the sand bed and a lot of filter cracks occur. There are two reasons for this.
* The sand specifications are very bad. The high coefficient of uniformity causes grading of the sand bed during backwash and consequently cake filtration and contributes to the forming of filter cracks and thus mud balls. The small sand grains (d_{10}=0.5mm) are hard to keep clean by backwashing with only water.
* Backwash rates are too low causing accumulation of dirt in the sand bed. One tank is emptied in about 15 minutes resulting in a backwash rate of 6 m/h instead of 60 m/h.

Disinfection

200 g HTH per day is dosed. This is 0.64 mg/l available chlorine, which is not enough. During the day the solution is diluted by adding water continuously while chlorine is added only once a day. This results in a concentration of chlorine that drops during the day. At the end of the day there is almost no chlorine in the solution. The water is not disinfected properly and is of a variable quality.
Mixing conditions are very poor because the disinfectant is dosed straight into the clear water tank. The retention time is long enough, 3 to 4 hours in the clear water tank.

Clear water tank

The capacity of 31 % of daily production is enough for the function it has at the moment, buffer between treatment and clear water pump. At the peak day for the gross 100% demand the capacity is 11% of the daily production which is probably just enough.

Clear water pumps

One large pump and one small one stand by. In case of a breakdown of the original pump there will be less water available in the town.
There are no spare parts at the plant, so when spares are needed it will take some time to get them. (There are spares in Choma, at the provincial water engineers office)

Clear water main

Head is unknown which makes it difficult to determine what pump is needed.
With d = 75 mm AC, $k = 1.0 * 10^{-4}$ and $\Delta H = 8.0$ m, Q is 11.9 m^3/h.
When the production rises in the future, this main will be insufficient. Larger lines should be placed.
There is a bulk meter in clear water main, so the production of clear water is known and the losses at the plant can be calculated (since there is a bulk meter at the intake too)

Service reservoirs

The total capacity is 31 % of the average daily production, this is just enough. For a peak day with the gross 100% demand the capacity is only 11 % of the daily production which is not enough.

Distribution

No mayor leakages in the reticulation, as far as it is known. This means that not much water is lost.
Only 32 % of town is connected to the water supply. This is a very small number. The number of illegal connections and might be high.
Furthermore a lot of people depend on wells or untreated river water for drinking and cooking. This gives higher risks for the common health of the people (think of cholera and diarrhoea for example).
The largest service mains have a diameter of 75 mm AC. Since Chirundu is a small town build on a river bank, the available head loss is considerable. When $\Delta H = 8$ m/km, Q = 11.9 m^3/h. This is not sufficient. It should be $2.2 * 22 = 48.4$ m^3/h for the gross 100% demand.

Conclusions

The present production, 9 m^3/h, is much smaller than the design capacity of the treatment which is about 30 m^3/h. Pre treatment is done in a up flow sludge blanket tank which does not work satisfactory at these low flow rates, especially not during the rainy season when alum is dosed.

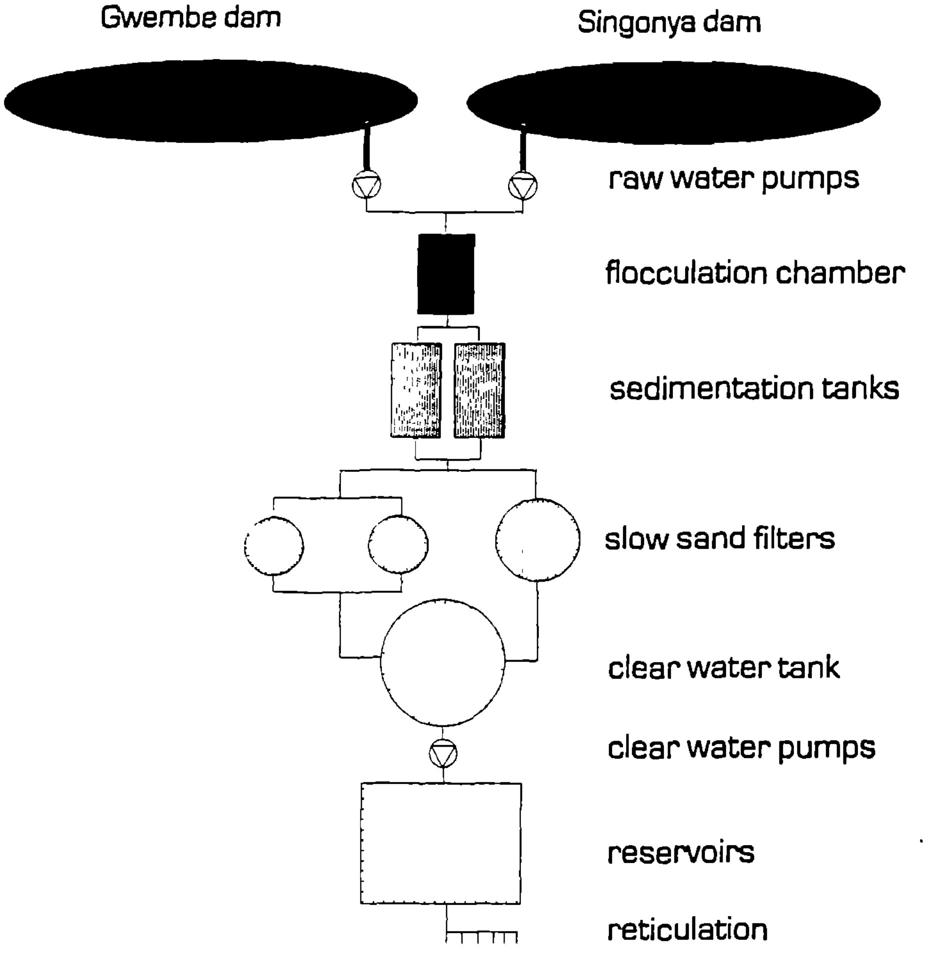
The capacity of raw and clear water mains and the distribution system on the other hand is much smaller, 10 to 12 m^3/h which is even smaller than the present gross demand. The filter control, except for some wastage of water, is all right. The sand characteristics and the backwash on the other hand are very bad. Sand grains are too small and there is not enough uniformity. The backwash does not produce enough expansion and scour. The result is filter cracks and loads of mud balls in the sand bed which cause a badly deteriorated filter operation.
The disinfectant dosage has not a constant concentration and mixing conditions are poor.

Recommendations

In the first place the filter sand and underdrainage need to be replaced by better sand and gravel. The backwash can be upgraded by using the two backwash reservoirs for one cleaning and enlarging the size of the pipe from the reservoirs to the filters, thus shortening the time needed for a backwash and increasing the backwash rate.
Best for the system is to renew the all the pumps and the raw and clear water main and increase the capacity to such an extent that it fits with the treatment, to about 30 m^3/h. Also the distribution system needs capacity expansion.
Of course a thorough study is required to show what exactly the alternatives are and which of them is the most economical and feasible.

CHIRUNDU CAPACITY SHEET		
Production now: 9 m^3/h	Demand now: 13 m^3/h	100 % Demand: average 22 m^3/h peak day 26 m^3/h peak hour 48 m^3/h
maximum	capacity in m^3/h	100% parameters
raw water pumps	13	200 % of the day
raw water main	11.3	dH = 43.9 m/km
sedimentation	16 (plain) 48 (flocculent)	s_0 = 1.64 m/h
filters	60	q = 4.4 m/h
clear water pumps	13	200 % of the day
clear water main	11.9	dH = 39.5 m/km
distribution	11.9	dH = 128 m/km

Gwembe



system layout

2.4 GWEMBE

Introduction

The water sypply scheme of Gwembe embodies two intake reservoirs at Gwembe and Siononga dam from wher the raw water is pumped to the treatment works. The treatment consists of a receiving chamber, two regtangular, horizontal flow, sedimentation tanks and three slow sand filters. Disinfection takes place at the clear water tank from where the water is pumped to two elevated reservoirs. From there the distribution to town takes place by gravity.

Intake of raw water

Gwembe dam. The capacity is unknown and there are said to be silt problems so as time goes by the capacity of the reservoir reduces. In this way it is not possible to calculate the maximum amount of water that can be abstracted during the dry season without new measurements of the present capacity.
The two raw water pumps are both broken at moment of visit.

Siononya dam. This dam seems to be in good conditions. One pump is working, the spare pump is broken at moment of visit, so in case of a breakdown of the working pump no water can be pumped to the treatment. The capacity of this pump is 51 m^3/h. The gross 100 % connection demand is 26 m^3/h (31 on a peak day), so the capacity of the pump is sufficient.

All pumps are electrical pumps and power comes from the Hydropower station Livingstone. During the rainy season regular breakdowns occur.
The diameter of the raw water main from Siononya dam to the treatment is 150 mm AC. When $k = 1 * 10^{-4}$ and $\Delta H = 4$ m/km than $Q = 51.7$ m^3/h.

Treatment

Floc forming and removal

Coagulation
During the rainy season approximately 5 kg of alum is dosed every day. It is added continuously during the day while the concentration of the solution stays the same. The dosage is varied depending on the sight of the raw water. One should do a Jar-test regularly in order to know the right amount of alum one has to dose.
The dosing is done on top of a weir where the water has a high velocity, so the rapid mixing seems to be sufficient.

Flocculation
This takes place in a baffle channel which seems sufficient.

Sedimentation
In the dry season, when no alum is added, plain sedimentation occurs which only removes coarser material. During our visit the turbidity of the water did not decrease in the sedimentation tank, it stayed about 33 NTU.
The total surface area of the sedimentation tank is 2 * 12.5 = 25 m^2. With an average daily production of 16 m^3/h, s_0 = 0.64 m/h now. The maximum surface load for plain sedimentation is 1 m/h which gives an capacity of 25 m^3/h. For flocculent settling, which also reduces the amount of suspended and colloidal matter, the maximum capacity will be 75 m^3/h.

Filtration

The design filter rate of the slow sand filters is 0.15 m/h. The average filter rate at time of visit was 0.16 m/h. The maximum capacity of 0.25 m/h for slow sand filters gives a maximum production of 50 m^3/h. The gross 100% demand is only 26 m^3/h.
The raw water pumps are working 16 hours a day, filters are running 24 hours a day. During the night the filters are still in operation but no water is added. Consequently the supernatant level and the filter rates will drop during night time.

Control
The filters are designed to be outlet controlled. A floater that controls the supernatant level on the filters is connected with the outlet valve. At time of visit this device was broken and the filter rate was controlled by opening the outlet valve by hand when the filter rate becomes too low. The filter rate can be determined with the V-notch in the effluent line of the filters. Thus it is easy to mistakes and as a result filter rates will vary during operation depending on the position of the outlet valve

Sand
The large filter has a sand layer thickness of only 15 cm, this is not enough because at least 70 cm is required. At some places at the filter bed surface even the under drainage is visible. As a result there will be no proper biological treatment and organic matter can pass the filter bed easily. Also short circuiting will occur, decreasing the effluent quality. The turbidity of the effluent during the visit was more than 13 NTU which emphasizes this.
The d_{10} of the sand is 0.3 mm, which is ok. The uniformity coefficient of 3.3 is a little bit high but will not give real problems.
The two small filters have a sand layer of approximately 60 cm so they need to be topped up also to about 1 m.

Cleaning
In order to clean the filters, they are completely drained and after this dried for a few days. Then the upper layer of dirt is removed from the filters after which the filters are taken in use again. This is not a proper procedure because the complete drying destroys the biological layer. It takes about two months to build up a new layer and until that time the produced water will not be biological reliable. While the cleanings are more frequent than once per two month, the biological processes will never work very well.

Disinfection

In the rainy season 1 kg of HTH is dosed daily, and in the dry season 0.6 kg. Since the production is 387 m^3/day, this gives a free chlorine dosage of 1.8 p.p.m. in the rainy season and 1.1 p.p.m. in the dry season. When the filters are working properly this could be just enough, but without proper biological treatment the dosage needs to be 2 to 3 p.p.m. always.
The dosing is done continuously direct into the clear water tank where mixing conditions are poor. The contact time between the water and the chlorine is long enough, about 4 to 5 hours in the clear water tank.

Clear water tank

The capacity is 110 m^3, which is 28 % of the daily production and 15 % of the peak day production at the gross 100 % connection demand. This is enough.

Clear water pumps

There are two working pumps of which one pump is stand by, this is good. $Q = 27.6$ m^3/h for both pumps. This is sufficient for the average gross 100 % connection demand which is 25.6 m^3/h, but for the peak day demand.

Clear water main

The size of this main is 150 mm diameter.
When $k = 1 * 10^{-4}$ and $\Delta H = 8$ m/km, then $Q = 73$ m^3/h. This is more than the peak day of the gross 100 % connection demand so more than enough.
The bulk meter in the clear water main is daily recorded. This is good because than one knows how much water is produced and what the water loses over the treatment are.

Service reservoirs

The total capacity of the reservoirs is 250 m^3. This is 64 % of the daily production now and 34 % of the peak day for the gross 100 % connection demand. This is enough to store the differences in production and demand over the day but not enough for fire fighting and supply during breakdowns.

Distribution

The big service mains have a diameter of 100 mm. When $k = 1 * 10^{-4}$, ΔH is 4 m /km than $Q = 17.9\ m^3/h$ for each main.
When, in case of the gross 100 % connection demand, the peak hour of a peak day has to be transported by the existing mains, dH will be 9.9 m/km. This is too much so expansion is required in this case.

GWEMBE CAPACITY SHEET		
Production now: 16 m^3/h	Demand now: 21 m^3/h	100 % Demand: average 26 m^3/h peak day 31 m^3/h peak hour 56 m^3/h
maximum	capacity in m^3/h	100% parameters
raw water pumps	51	61 % of the day
raw water main	52	dH = 1.5 m/km
sedimentation	25 (plain) 75 (flocculent)	$s_0 = 1.24$ m/h
filters	50	q = 0.16 m/h
clear water pumps	28	111 % of the day
clear water main	73	dH = 1.5 m/km
distribution	36	dH = 9.9 m/km

Conclusions

The capacity of the raw water reservoirs are not known, so it is not possible to calculate the maximum draw off for the dry season and to check if the reservoirs are large enough to increase the production.
Except for the distribution, all the components of the system, if operated very good, should be able to handle the gross 100% demand.
During the rainy season, when alum is dosed, no Jar tests are carried out to determine the best alum dosing. This is why the floc forming probably will never be optimal(and also the sedimentation not).
The automatic filter control device, a floater mechanism, is broken and the filter control is done by adjusting the resistance over the effluent valve by hand. While this is done irregular the filter rate will be varying.

The sand bed in the filters is too thin, one only has 15 cm of sand left with at some places the under drainage coming through the surface. Consequently a lot of material, organic matter and suspended and colloidal particles, will pass the filter bed, thus deteriorating the effluent quality.
The cleaning procedure causes deterioration of the biological layer while with every cleaning the filter bed is dried completely. Consequently the biological clarification is not very good.
Mixing conditions in the clear water tank for dosed chlorine are poor causing unequal distribution of disinfectant over the clear water.

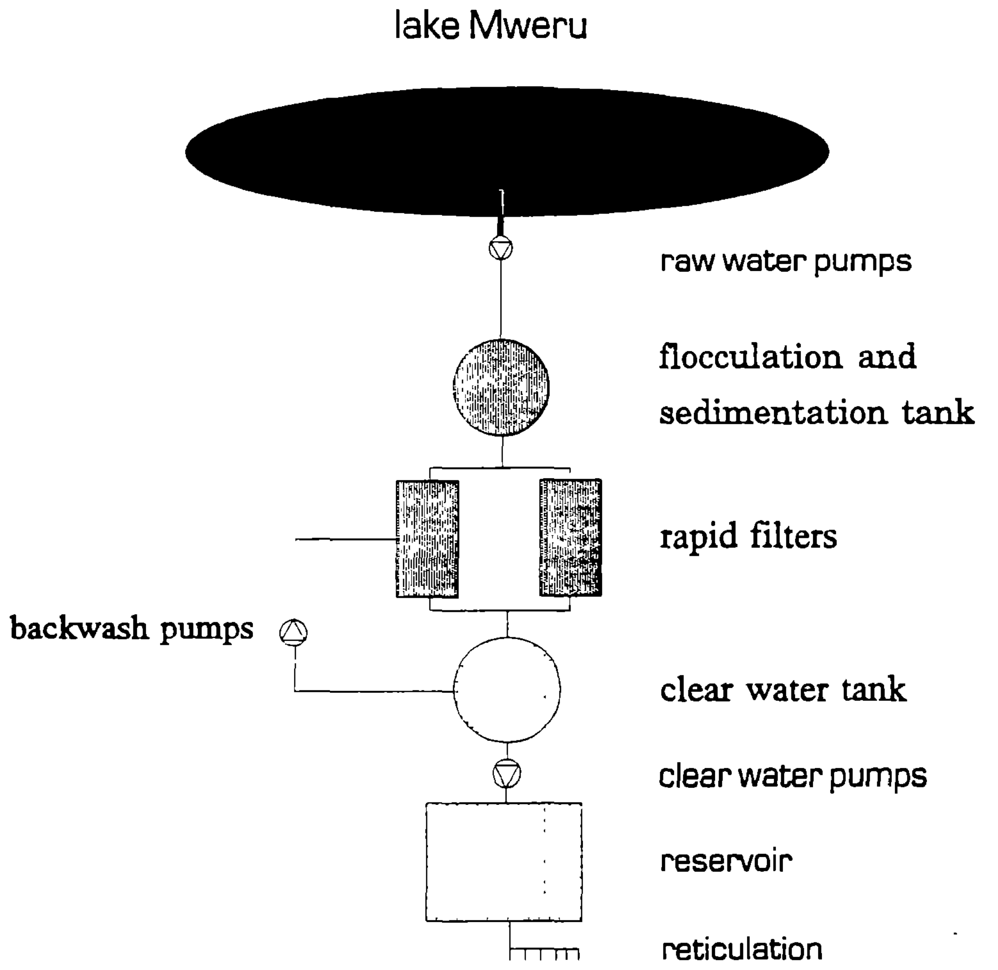
Recommendations

In order to know the maximum draw off of raw water for the dry season, the capacity of the two reservoirs should be determined.
For optimal pre treatment jar tests should be carried out regularly in order to know the right amount of alum to be dosed.
The filters should be filled to such an extend that is has about 1 m of sand layer again and the filter control device with the floater should be repaired. The biological layer can be kept intact by not drying the filter bed completely during cleanings, but just draining it to about 0.2 metres under the surface of the bed as is necessary for manual cleaning.
The cleaning procedure than changes to the following:

a. drain the filter to approx. 0.2 metres beneath the sand surface
b. directly after this scrape off the upper 2 to 3 cm of the sand bed
c. refill the filter again by backfilling.

The filters than will work satisfactory with a good effluent quality.
Some proper mixing conditions should be realised for the disinfectant, for example dosing at the inflow of filtered water into the clear water tank.

Kashi Kishi



system layout

2.5 KASHI KISHI

Introduction

The water supply scheme of Kashi Kishi embodies an intake from lake Mweru from where the raw water is pumped to the treatment works. The treatment consists of one circular, combined, floc forming and sedimentation tank of the accelator type, two rapid filters and one back wash pump. Disinfection takes place at the clear water tank from where the water is pumped to the elevated reservoirs. From there the distribution to town takes place by gravity.

Intake of raw water

Water is taken in from the large lake Mweru so quantity problems are not expected. Due to the fact that the water comes from a big lake, the water quality is relatively constant which is positive for the treatment.
There is only one raw water pump. No spare pumps are there, so when this pump breaks down, no water will be pumped into the treatment and thus the town is without water. At time of visit the raw water pump was just repaired, after three month without water, but new problems occurred already after one day.
The raw water main has a diameter of 150 mm GI. When $k = 1.5 * 10^{-4}$ and $\Delta H = 4$ m/km then $Q = 49$ m^3/h. This is not enough for the gross 100 % connection demand which is 83 m^3/h.
All pumps are electrical pumps and power is supplied by Zesco. Especially during the rainy season regular breadowns occur.

Treatment

Floc forming and removal

No alum is dosed for the last 15 years, this means that there are no coagulation and flocculation processes.
The combined sedimentation tank, accelator type, is a circular tank with a smaller circle in the middle where the coagulation and flocculation are supposed to take place. The mechanical mixer is broken down so if alum would be dosed, no proper mixing would occur.
A scraper in the tank is supposed to remove the sludge in the sedimentation tank, this is broken to so sludge has to be removed by hand.
The sedimentation tank has an area of 55.4 m. At time of visit the surface load s_0 was 0.56. The maximum capacity of the sedimentation tank is 55.54 m^3/h in case there is no alum dosed(plain sedimentation), and 165 m^3/h when alum is added in the right dosage

and there is proper coagulation and flocculation.
The circular overflow weir over which cleared water is carried off is not exact horizontally placed in the tank so the water that flows over is not equally shared over the whole circle. The velocity of the water varies over the tank and a kind of short-circuiting in the sedimentation tank occurs which influences effluent qualities.

Filtration

Control
The rapid filter control is inlet controlled. The rising supernatant level gives pressure to overcome the rising resistance due to clogging of the filter bed, so the filter rate stays constant.
The production (when the system is working) is about 30 m^3/h which gives an average filter rate of 2 m/h. Even with this low filtration rate their are still complaints that sometimes the filters can not handle the flow. The maximum capacity of the filters, when the average maximum filter rate is 10 m/h, is 145 m^3/h.

Sand
The uniformity U = 4.59 is much too much. During backwash this low uniformity causes grading of the sand bed. The d_{10} = 0.37 mm, is much too small for rapid filtration. Together they cause cake filtration, low filtration rates, mud balls and filter cracks.

Backwash
The backwash pumps are not working, only the air backwash is possible. At time of visit it was not clear since when the pumps were broken (contradictory statements). Also the sand used at the moment is impossible to backwash with good results, grading will occur and there will not be enough scour. For example the d_{50} of 1.52 mm requires imaginary backwash rates of about 100 m/h which is 720 m^3/h, but at these rates all the smaller grains will be washed out of the sand bed.

Disinfection

Every day 1 kg of HTH powder is dosed in the clear water tank, when available. This is an average of 0.94 p.p.m. which is not enough.
The contact time is about several hours only in the clear water tank.
Mixing conditions are poor in the clear water tank and dosing is not done continues.
Consequently a lot of not disinfected water is distributed with all the dangers for the consumers. This is emphasized by the yearly cholera outbreak in the rainy season.

Clear water tank

The tank has a capacity of 465 m^3, this is 62 % of the average daily production and 20 % of the peak day for the gross 100% demand. This is more than enough for the functions it can have.

Clear water pumps

There is only one clear water pump. When this pump breaks down, there will be no distribution of water to town. The supply guaranty is very low here.

Clear water main

The main has a diameter of 150 mm AC. When $k = 1 * 10^{-4}$ and ΔH is 4 m/km, than $Q = 52$ m^3/h.

Distribution

The reservoirs are located at the secondary school, 280 m^3 which is 12 % of the peak day production for the gross 100% demand. This is too small to store differences between demand and production over the day.
Part of the distribution is done directly from the clear water main, part is distributed from the reservoirs.
The big service main has the same diameter as the clear water main, 150 mm, with the same maximum capacity, 52 m^3/h.

Conclusions

Because there are no spare pumps and the pumps that are used are not in a very good condition, the township is frequently without water. For example at time of the visit there had not been water for three month.
The design capacity of the treatment is very high compared with the present production and even compared with the gross 100% demand. The capacity of the pumps and mains on the other hand is just enough for the present gross demand.
The treatment plant is old, about 25 years, and has deteriorated badly. The settling tank, including coagulation and flocculation, and all its devices (sludge scraper, mechanical mixer etc.) are broken down and water is just flowing through the tank without any result. Proper coagulation, flocculation and sedimentation is not possible here. The rapid filters are even worse. The sand that is used has completely wrong characteristics and backwash pumps are broken(only the air backwash is working). The result is grading of the sand bed, cake filtration, filter cracks, mud balls, accumulation of dirt,blockage of the filter bed and very low filter rates.

Disinfection is not done continuously and the dosages are too low, so partly or not at all disinfected water is distributed with all the dangers for the consumers. The yearly cholera outbreak in Kashi Kishi may, with others, be one of the results.

Recommendations

A thorough study is required in order to give advice about what has to be done with the water supply system. Especially for the present treatment it doubtful if it is economic and feasible to rehabilitate it.
There is also no good temporary solution. The only thing is to replace the sand of the filters for sand that meets with the right specifications, repair the backwash pumps(or place a backwash reservoir on top of the control room) and use the treatment for the time it is necessary. The pumps are not reliable at the moment so spare ones are required
Important is to increase the active chlorine dosing which is 0.7 * amount HTH to be dosed to about 2 or 3 p.p.m.. This means a HTH dosage of 3 to 4 grams per m^3 produced water. A dosing device is required to dose continuously and good mixing conditions need to be introduced.

KASHI KISHI	CAPACITY SHEET	
Production now: 31 m^3/h	Gross demand now: 49 m^3/h	Gross 100 % Demand: average 83 m^3/h peak day 99 m^3/h peak hour 178 m^3/h
maximum	capacity in m^3/h	100% parameters
raw water pumps	?	
raw water main	49	dH = 16 m/km
sedimentation	55.4 (plain) 165 (flocculent)	s_0 =1.79 m/h
filters	145	q = 6.9 m/h
clear water pumps	?	
clear water main	52	dH = 14.7 m/km
distribution	52	dH = 47 m/km

2.6 LUNDAZI

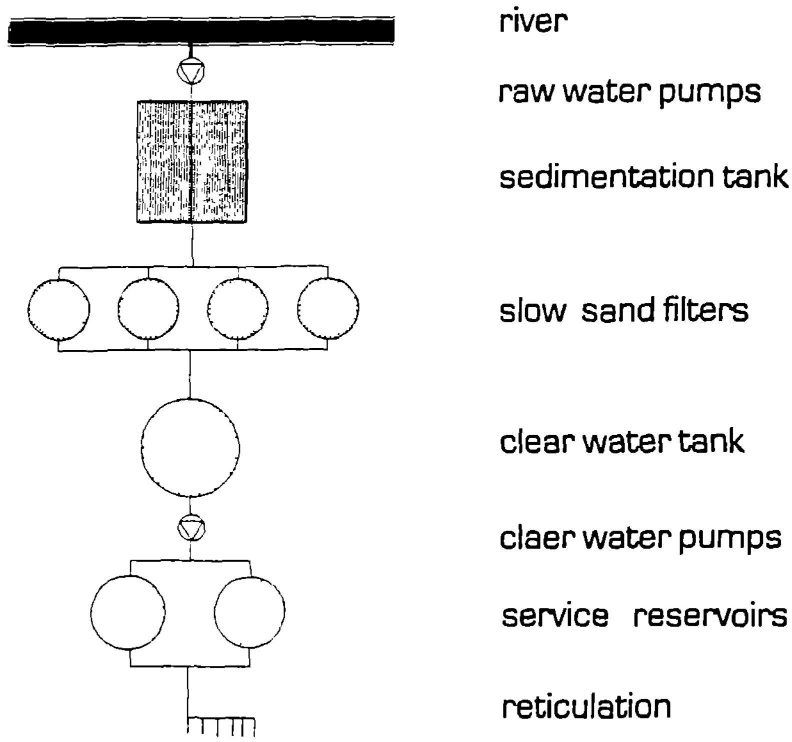
Introduction

The water supply scheme of Lundazi embodies an intake directly from the river (no reservoir) from where the raw water is pumped to the treatment works. The treatment consists of one regtangular, horizontal flow, sedimentation tank and four slow sand filters. Disinfection takes place at the clear water tank from where the water is pumped to two elevated service reservoirs. From there the distribution to town takes place by gravity.

Intake of raw water

In 1986 the reservoir dam is washed away so there is no raw water storage reservoir at the moment and Lundazi completely depends on the basic flow of the river. Every year there is a shortage of water in the dry season. At the year of visit ('92) there was already almost no water in the river anymore at the end of June. The normal basic flow of the river is unknown.
The intake device just lays on the bottom of the river so sludge is taken in.
Because the water is taken directly from the river, the quality and turbidity vary a lot. In the rainy season the raw water has a high turbidity.
Normally there are three raw water pumps with three separate raw water mains to bring the water to the treatment. Two of the pumps were not working at time of visit so there were no spare pumps. When the raw water pump breaks down, no water can be pumped into the treatment any more.
The raw water pump has capacity of 114 m^3/h which is much more than the gross 100 % connection demand of 72 m^3/h.
The raw water main has a diameter of 150 mm. With $k = 1.5 * 10^{-5}$ and $H = 4$ m/km, the capacity is 49 m^3/h. For both the gross demand now as for the gross 100 % connection demand this is not enough.
The raw water pump is not suitable for the raw water main. Its capacity decreases because the resistance over the main is very large. Furthermore cavitation might occur which can damage the pump.
The bulk meter in the raw water main is not working so the amount of water abstracted from the river is not known.
All pumps are electrical pumps and since there is a new generator the supply of power is good.

Lundazi



system layout

Treatment

Floc forming and removal

Coagulation and flocculation
Since alum is dosed straight into the receiving chamber of the sedimentation tank where there is almost no turbulence, rapid mixing does not take place. Because the alum dosing device is broken the total daily amount of alum is added at once. A flocculation chamber is not available. A flocculation chamber is even not there. The result that there is almost no floc forming with all the consequences for the sedimentation.

Sedimentation
As there is no proper coagulation and flocculation there will be only plain sedimentation which removes only coarse relatively material.
With a surface area of 72 m^2 the average production of 29 m^3/h gives a surface load of s_0 = 0.4 m/h. For plain sedimentation the maximum surface load s_0 = 1 gives a maximum production of Q = 72 m^3/h. If proper coagulation and flocculation are introduced flocculent settling will occur, giving a maximum capacity of 216 m^3/h.
At the inlet water is distributed equally over the width of the tank but the outlet of the tank is designed wrong because it is situated at one point and not over the whole width. This causes short circuiting and eddies. The ratio between length and width is 4 (3*12m each tank) which is good for a stable flow.

Filtration

Control
The filters are designed as 'inlet controlled filters'. This requires a constant inflow from the sedimentation tank and thus from the raw water pumps. Instead the raw water pumps are operated depending on the supernatant level in the filters. This level is kept high by starting the raw water pumps as they drop too far and stopping them as the filters start overflowing. So the filters are operated with declining rate filtration, giving the least clogged filter most of the water. Consequently the filters clog too fast, especially during the first few days after cleaning, which shortens the filter runs (or discharge) considerably. The average filter rate at time of visit is 0.08 m/s. The maximum capacity, with an average filter rate of 0.25 m/h is 4 * 0.25 * 176 is 176 m^3/h. This is enough, even for the gross 100 % connection demand.

Sand
A red line on the wall of the filter indicates the sand level where refill of sand is needed. In this way one always knows whether there is enough sand in the filter bed or not. This is very useful, as long as the operators are aware of the meaning of the red line and money for new sand can be found.
The coefficient of Uniformity, U = 4.21, too high for a slow sand filter
D_{10} is 0.28, this is good.

Cleaning
Before cleaning the filter is dried completely, which destroys the biological layer in the filter bed. As a result the biological treatment will never be optimal because it takes about two month to built up a new biological layer, which is longer than the period within two cleanings.

Disinfection

The dosing device for the HTH powder is broken. Now twice a day 300 g of HTH powder is dosed directly into the clear water tank where mixing conditions are very poor. This does not work very well since one should dose continuously in order to get the right concentration of free chlorine in all the produced water. The average dosage is 0.6 p.p.m. of active chlorine which is not enough.

Clear water tank

The capacity is 454 m^3. This is 65 % of the daily production and 22 % of the peak day for the gross 100% demand. This is enough.

Clear water tanks

No stand by pumps were available at time of visit. This is not good since no water can be delivered to town when the clear water pump breaks down.
The capacity of the pump is approximately 50 m^3/h. This is not enough for the gross demand now.

Clear water amin

The size of this main is 200 mm AC.
When $k = 1*10^{-4}$ and H is 4 m/km than Q = 110 m^3/hour, which is enough for the gross 100 % connection demand.

Service reservoirs

The capacity is 77 % of the daily production, which is more than enough to store the differences in production and demand over the day, for fire fighting and for supply during breakdowns. But for the peak day at the gross 100% demand the storage is only 26 % of the daily production, which is not sufficient fot the above mentioned functions.

Distribution

The distribution starts with two 80 mm AC mains. When $k = 1 * 10^{-4}$ and $\Delta H = 4$ m/km than $Q = 10$ m³/h for each main. This is much too small, even for the present production. The only thing is that the information we obtained about the diameters could be wrong, because this calculated capacity is very low.
When the peak hour of a peak day has to be transported in these mains, in case of the gross 100 % connection demand, the total head loss will be 242 m/km. This is impossible.
The connection percentage of the water supply system in Lundazi is very high, so probably the illegal draw off will be very low.

LUNDAZI CAPACITY	SHEET	
Production now: 29 m^3/h	Gross demand now: 68 m^3/h	Gross 100% Demand: average 72 m^3/h peak day 86.4 m^3/h peak hour 155 m^3/h
maximum	capacity in m^3/h	100% parameters
raw water pumps	114	84 % of the day
raw water main	49	dH = 12.3 m/km
sedimentation	72 (plain) 216 (flocculent)	$s_0 = 1.2$ m/h
filters	176	q = 0.12 m/h
clear water pumps	50	173 % of the day
clear water main	110	dH = 2.5 m/km
distribution	20	dH = 242 m/km

Conclusions

Because there is no raw water reservoir (reservoir dam is washed away in 1986) the water supply depends on the basic flow of the river which was already almost zero in June 1992. Months without water at the end of the dry season already occurred.
The intake device looks provisional, is vulnerable and also sludge is taken in with it because it just lies on the river bottom.
The capacity of the raw water pump is much too high compared with the clear water pump. The result is that the raw water pump is constantly switched on and off.
There almost no floc forming because the daily amount of alum is added at once, there is no rapid mixing and even no flocculation chamber. This reduces the sedimentation to

plain settling which removes only relatively course material.
The filters are operated with declining rate filtration with an almost constant supernatant while they are designed as constant rate filters with a rising supernatant level. One of the causes is the high capacity of the raw water pumps which make it impossible to pump a constant flow lower than its capacity into the treatment.
The cleaning procedure of the slow sand filters causes deterioration of the biological layer while with every cleaning the filter bed is dried completely. Consequently the biological clarification is not very good.
The disinfectant, HTH, is not dosed continues and mixing conditions are poor. Also the average amount dosed is too low giving risks for the consumers.

Recommendations

The most urgent thing is to build a new raw water reservoir for supply during the dry season.
To introduce a proper floc forming process, a flocculation chamber and good mixing conditions for the coagulant are required.
For the filters, the biological layer can be kept intact by not drying the filter bed completely during cleanings, but just draining it to about 0.2 metres under the surface of the bed as is necessary for manual cleaning.
The cleaning procedure than changes to the following:

a. drain the filter to approx. 0.2 metres beneath the sand surface
b. directly after this scrape off the upper 2 to 3 cm of the sand bed
c. refill the filter again by backfilling.

The filter control can only be changed if raw water pumps with a smaller capacity or when clear water pumps with a higher capacity are used. Taken in account the future demand it is better to place clear water pumps with a higher capacity.

For the disinfection a proper dosing device should be placed and proper mixing conditions should be introduced, for example at the inflow of filtered water into the clear water tank. The dosage should be increased to about 3 to 4 grams of HTH powder per m^3 produced water.

2.7 MANSA

Introduction

The water supply scheme of Mansa embodies an intake directly from Mansa river (no reservoir) from where the raw water is pumped to the treatment works. The treatment consists of only four slow sand filters. Disinfection takes place at the clear water tank from where the water is pumped to two elevated service reservoirs. From there the distribution to town takes place by gravity. A part of the water is distributed directly from the clear water main.

Intake of raw water

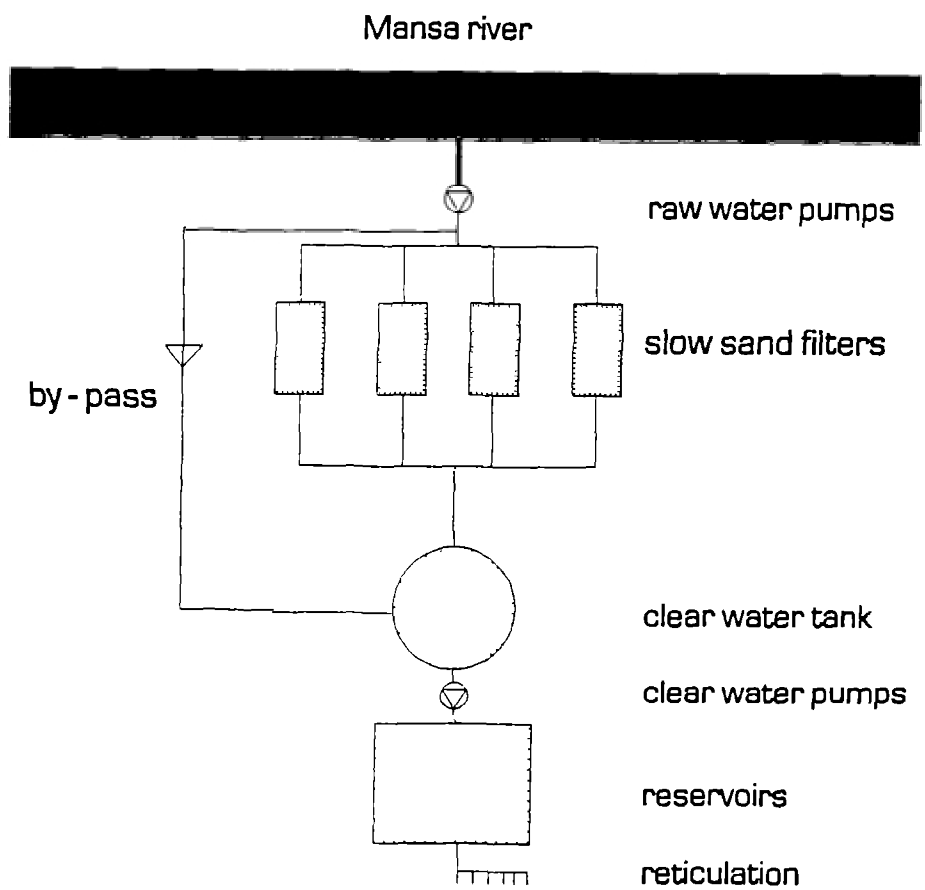
Water is directly drawn from the river, without any reservoir. This means the water production is depending completely on the basic flow of the river. Normally this does not give problems but at time of visit (extremely dry year) low levels started to give problems. In order to raise the water level at the end of the dry season a small weir from stones and sand bags is constructed every year. It is doubtful if this provisional construction is enough this year.
No flow measurements are done. This means the exact amount of water that is taken in is unknown. There are 3 raw water pumps, two of 150 m^3/h and one of 192 m^3/h also there is a spare pump as stand by for break downs. When one of the smaller pumps is used as spare pump, the maximum capacity of the raw water pumps is 342 m^3/h, which is enough for the gross demand now, but not for the gross 100 % connection demand.
There are three very short raw water mains with sizes of 150 and 200 mm. With $k = 1.5 * 10^{-4}$, $d = 150$ mm and $H = 8$ m/km, Q will be 70 m^3/h. With $k = 1.5 * 10^{-4}$, $d = 200$ mm and $H = 8$ m/km, Q will be 148 m^3/h.
All pumps are electrical pumps and power is supplied by Zesco. Especially during the rainy season ragular breakdowns occur.

Treatment

Floc forming and removal

There is such thing. Because the water is taken in directly from the river, the turbidity will be varying during the year and especially during the rainy season raw water turbidities can become very high. Since slow sand filters are used as water treatment these high turbidities can give problems.

Mansa



system layout

Filtration

There are four slow sand filters, A = 540 m^2 each, total area 2160 m^2.

Control
The filters are designed as inlet controlled and used like that.
Filter runs are about three days, which means that after three days there is almost no discharge anymore. Because it is impossible to clean the filters every three days, overflowing of the filters occurs often, especially during the rainy season. Main reason for this is that there is no pre treatment so high turbidity loads on the filters will regularly occur. Another reason is that the sand that was still used at time of visit was very dirty.
The maximum capacity of the filters is 540 m^3/h. The gross water demand now is 332 m^3/h, so the capacity is sufficient for that. The gross 100 % connection demand is 726 m^3/h (average, 871 peak day), in that case the capacity of the filters is too small.
Less than 50 % of the produced water is not filtered but directly from the river mixed up with filtered water and pumped into the clear water tank. It is said that this is only temporary while the filters are rehabilitated. The average filter rate in this case is less than 0.12 m/h.
Only two filters are used at the same time now.

Sand
The uniformity U = 2.46 and d_{10} = 0.37 (of the new sand) and the sand layer thickness is 1 m, this is all right. Only the sand used at the moment is very old, about 15 years, and not very clean which can cause too much resistance, and only now they are renewing it. Normally, with a filter run of about one and a half month, a sand bed is renewed every two years because with every cleaning a few centimetres of sand are removed.

Cleaning
In the dry season cleaning is done by draining the filters completely, drying the filter bed by the sun and than removing the upper layer of dirt. In this way the biological layer is destroyed with every cleaning.
In the rainy season the filters clog so fast that there is no time to dry them. Then a biological layer can be formed again. This is emphasized by the fact that in the beginning of the rainy season the treated water is coloured but after one or two months treated water is colourless. This colour is caused by biodegradable material which, when the biological treatment works sufficient is removed. Probably, with cleanings a too thin sand layer is removed because the sand used at the moment is much too old, causing accumulation of dirt and consequently too low filter rates and too short filter runs. New sand was brought in at time of visit.

Construction
The filters are full of cracks that cause leakages. This is why a lot of water gets lost. At the time of visit the filters were being rehabilitated.

Quality
The turbidity of the filtered water at time of visit was 1.2 NTU which is very good.

Disinfection

2 kg of HTH powder is added continuously into the clear water tank every day. This is 0.22 p.p.m. of active chlorine. This is not enough, especially not since part of the water is untreated.
At time of visit 10 kg of HTH was in stock . This means there will be a shortage very soon.
Since the chlorine is dripping directly into the clear water tank, the mixing are very poor. The contact time time for the chlorine is long enough, about 2 to 4 hours in the clear water tank.

Clear water tank

The capacity of the tank is 18 % of the daily production but only 5.5 % of the peak day for the gross 100 % connection demand. 5.5 % means about one hour of production and is not really much.
The turbidity of the water in the clear water tank is 3.4 NTU. This water has a higher turbidity than the filtered water because 192 m^3/hour of unfiltered water is coming in together with 150 m^3/h of filtered water.

Clear water pumps

At time of visit there were no stand by pumps. In case of a breakdown of the used pump, no water is distributed to town.
Q = 270 m^3/h, this is 194400 m^3/month. This is not sufficient since both the gross water demand now and the gross 100 % connection demand are higher.

Clear water main

The main has a diameter of 300 mm GI, the length is 1.2 km.
When $k = 1.5 * 10^{-4}$ m and ΔH is 4 m/km, than Q = 302 m^3/h. This is enough now but not sufficient for the gross 100 % connection demand.
The non return valve in the main is leaking badly so water is wasted.

Service reservoirs

The capacity of the reservoir is 1388 m^3 . This is 21 % of the daily production and 6.5 % of the peak day for the gross 100 % connection demand. This capacity is really not enough, which is emphasized by the fact that when the outlet valve is opened in the morning the full tank is emptied within a few hours.

Distribution

The water distribution takes place from 06.00 till 18.00 hours. High parts of town get water only from 6.00 till 10.00 hours. This means that at 10.00 hours the reservoirs in town are already empty and there is only delivered water directly from the clear water main.
Only about 22 % of Mansa's population is officially connected to the water supply system (according to connection figures of the District Council and population figures of Central Statistics), so a really high illegal draw off is expected. The present production equals more or less the present demand based on connection figures plus 15 %, and still there is a big shortage which emphasizes the last statement.

MANSA CAPACITY	SHEET	
Production now: 270 m³/h	Gross demand now: 332 m³/h	Gross 100 % Demand: average 726 m³/h peak day 871 m³/h peak hour 1568 m³/h
maximum	capacity in m³/h	100% parameters
raw water pumps	342	255 % of the day
raw water main	366	dH = 45 m/km
sedimentation	n.a.	n.a.
filters	540	q = 1.61 m/h
clear water pumps	270	323 % of the day
clear water main	302	dH = 33 m/km
distribution	302	dH = ? m/km

Conclusions

The Mansa water supply system can just handle the Gross demand based at the present connections, but in no case the gross 100% demand.
The discharge of the river from which raw water is directly abstracted starts to give problems at the end of the dry season every year (low levels etc.). It is doubtful if the discharge is enough for the gross 100% demand without building a reservoir.
During the visit more than 50 % of the produced water was pumped directly without any treatment into the clear water tank, with all the consequences for the water quality of the distributed water.

Even though raw water is directly taken from a river it is not pre treated and put on slow sand filters. The resulting problems are very serious. The high turbidity loads cause slow sand filter runs of about 3 days and filters are frequently overflowing.
The sand layer that is scraped off during cleanings is too thin because the sand still used at time of visit was about 15 years old while with normal operation the complete sand bed is replaced in about two years. This causes accumulation of dirt in the sand bed and consequently low filter rates and short filter runs. At time of visit the old sand was being replaced by new which is very good for the filter operation.
Also the cleaning procedure causes deterioration of the biological layer while with every cleaning the filter bed is dried completely. Consequently the biological clarification is not very good.
The chlorine dosage is too low and mixing conditions are poor, causing even not disinfected water to be distributed.

Recommendations

A study should be carried out for expansion of the water supply system because the present capacity is not enough.

To be able to run the slow sand filters in a decent way pre treatment, for example floc forming and removal, of the raw water is indispensable.

The accumulation of dirt in the top layer of the sand bed can be prevented by scraping off the upper five centimetres of the sand bed with every cleaning. The complete sand bed is then renewed within about two years.

The biological layer can be kept intact by not drying the filter bed completely during cleanings, but just draining it to about 0.2 metres under the surface of the bed as is necessary for manual cleaning.
The cleaning procedure than changes to the following:

a. drain the filter to approx. 0.2 metres beneath the sand surface
b. directly after this scrape off the upper 2 to 3 cm of the sand bed
c. refill the filter again by backfilling.

Increasing the dosage of HTH powder to about 3 or 4 grams per m^3 produced water is necessary in order to distribute safe water.

2.8 MWENSE

Introduction

The water supply scheme of Mwense embodies an intake directly from Mwense stream (no reservoir) from where the raw water is pumped to the treatment works. The treatment consists of a receiving chamber, two circular, combined, upflow floc forming/sedimentation tanks that work with a sludge blanket, two rapid filters and four back wash tanks. Disinfection takes place at the clear water tank from where the water is pumped to the elevated service reservoirs. From there the distribution to town takes place by gravity.

Intake of raw water

Water is abstracted from Mwense stream, with a base flow 43.2 m^3/h
The gross 100 % connection demand is 43 m^3/h, so without a reservoir the capacity of the stream is not sufficient.
At the intake site a provisional weir to rise the water level is built. The structure is in a bad condition and seepage occurs. At time of visit the short intake channel from the river to the pump house was not in use because the water level was too low. Instead a hose with a strainer on the end was lying on the river bottom as intake device which causes mud to be taken in with the raw water.
Until february 92 a borrowed flight pump was used with a capacity of 220 m^3/h. The capacity of this pump was much too high for the water supply system of Mwense, and regular pipe bursts in the raw water main occurred. Now a KSB pump with a capacity of 15 m^3/h is used, which is too small.
The raw water main, 150 mm AC, length 900 m. When $k = 1 * 10^{-4}$ and $\Delta H = 4$ m/km, then $Q = 52$ m^3/h This is more than the gross 100 % connection demand.
All pumps are electrical pumps and power is supplied by Zesco. Especially during the rainy season regular breakdowns occur.

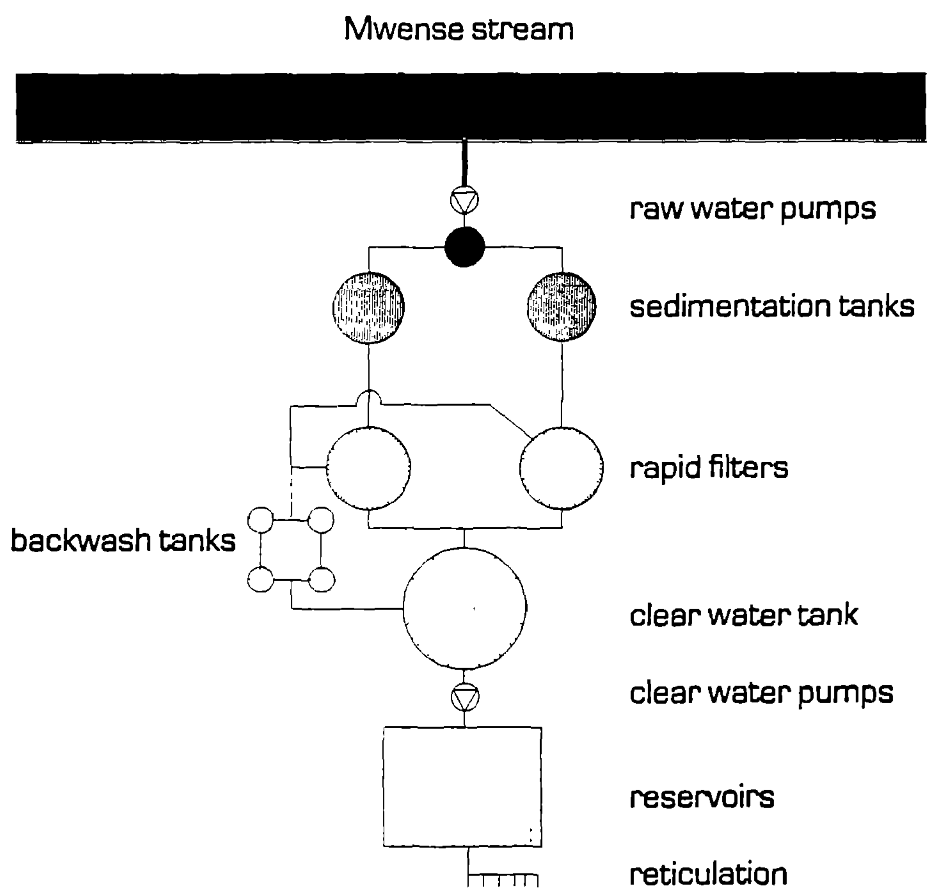
Treatment

The treatment consists of two identical lines, each with a pre treatment tank and a rapid filter. Only one line was used at time of the visit.

Floc forming and removal

Alum is dosed only in the rainy season when the turbidity of the water is high. This is ok when the turbidity in the dry season is low enough.

Mwense



system layout

In the rainy season twice a day 5 kg of alum is said to be dosed straight into the receiving chamber where raw water enters the treatment. Coagulant should be dosed continuously with a constant concentration, the way it is now results in much too high coagulant concentrations just after dosing and probably concentrations of about zero in between dosings. Good coagulation depends on the right amount of alum. This amount should be determined by executing a jar test every day or week (depending on the variation of the turbidity). When operators in charge have a lot of experience, dosing on sight can be sufficient.
In the rainy season it is said that there is only delivered 50 kg of alum a month. This means that there is enough alum for only 5 days a month!

The sedimentation tank is of the circular 'sludge blanket' type as explained in the chapter 'theory' of this appendix. With surface loads between 1 and 3 m/h, Q between 16 and 48 m^3/h (for flocculent settling which only occurs with proper coagulant dosing), the rapid mixing down in the tank and the flocculation in the cone are sufficient.
At the plant operators expect the sludge to settle at the bottom where no drain can be found and at times empty the tank manually to remove the sludge.
When no alum is dosed the tank is used as a plain sedimentation tank. With a surface area $A = 15.9$ m^2 and an average production of 20 m^3/h, the surface load $s_0 = 1.26$ m/h, this too high for plain sedimentation and makes the whole sedimentation not very effective. For the surface load $s_0 = 1$, the maximum Q is found: Q= 15.9 m^3/h, which makes the capacity for the two treatment lines together 32 m^3/h. During the rainy season there is no proper coagulant dosing so also the flocculant settling will not be very effective.

Filtration

Control
An overflow weir in the effluent line to the clear water tank protects the rapid filters against negative pressures. This is very useful for good operation. Filter control is done on sight: when the water level rises too high and the filters overflow, or when the effluent looks dirty, the back washing procedure is started. At time of visit the turbidity of the raw water was 16 NTU and after filtration it was 17 NTU while the filter rate was very low (1.64 m/h) which emphasizes the wrong operation.
The system is designed to be inlet controlled with a rising supernatant level and is treated like that, only backwashing should start before the filters overflow and water is wasted. The design filter rate is 5 m/h, at time of visit a rate of 1.64 m/h was found. The average filter rate is 3.4 m/h. A maximum average filter rate of 10 m/h, gives a maximum production of 60 m^3/h for each filter, and 120 m^3/h in total for the two lines. The design rate gives a capacity of 30 m_3/h for each filter, 60 m^3/h in total.
The tanks themselves are in bad condition, rust occurs everywhere and the construction is very old.

Sand
The uniformity of the sand, U = 2.0, is to high which causes grading during backwashing. The effective size of the sand grains d_{10} = 0.76 mm is pretty small what makes it hard to produce enough scour with backwashing in order to keep the sand clean.

Backwash
Four backwash tanks of 9 m^3 each are built on top of the office, 6 to 8 m above the water level in the filter. For backwashing of a filter only two tanks for one cleaning are used. While the filter bed area is 6 m^2 a backwash reservoir with a capacity between 18 and 36 m^3 placed at least 4 m above the water level in the filter is required. Thus the present backwash facilities seem all right. The sand used, with d_{50} = 1.40, requires a backwash rate of about 80 $m^3/m^2/h$ (T = 20°C), or 480 m^3/h for the whole filter, in order to get enough expansion and scour to keep the sand bed clean. The two backwash reservoirs thus need to be emptied each within 70 seconds making the total backwash time 140 seconds. The present situation is not like that, backwashing takes about 30 minutes so backwash rates are too low. This results in accumulation of dirt, filter cracks and mud balls and consequently lower filter rates and shorter filter runs. Filter runs are still not very short in Mwense, about 2 days in the rainy season, but this is probably because the production and thus the filter load is relatively low.

Disinfection

Each day 2 kg HTH powder is dosed in four shifts of 500 g, by mixing it with water and adding it continuously in to the clear water tank. This means 3 p.p.m. is added, this is good.
The mixing conditions in the clear water tank are not very good.
The contact time in the clear water tank is long enough, several hours.

Clear water tank

The capacity of the clear water tank is 450 m^3. This is 96 % of the daily production now and 36 % of the peak day for the gross 100 % connection demand. This is enough to store the differences between demand and production during the day, but not much is left for fire fighting and supply during breakdowns.
There are leakages in the clear water tank due to cracks in the walls. Drinking water is wasted and the foundation of the tank might get damaged. There are some big holes in the clear water tank. Birds and mosquitos might come in the tank and pollute the water.

Clear water pumps

There are no spare pumps stand by. This means that if the pump breaks down, no water is distributed to town.
The capacity of the pump is 24 m^3/h. This capacity is too small for both the gross demand now and for the gross 100 % connection demand.

Clear water main
The diameter of the main is 150 mm GI. If $k = 1.5 * 10^{-4}$ and $\Delta H = 4$ m/km then Q = 49 m^3/h. This is more than the gross 100 % connection demand.

Service reservoirs

The total capacity of the reservoirs in town is 74 % of the daily production now and 27 % of the peak day for the gross 100 % connection demand. This is just enough to store the differences in demand and production over the day but not for fire fighting and supply during breakdowns.

Distribution

Service mains
At times that the right size of a main, that has to be replaced, is not available, it is said to happen that a bigger size of main is placed. This is not good because the sudden increase of the diameter of the main causes turbulence and a pressure drop. Furthermore the velocity of the water decreases so that floating particles settle down.
The big service mains have a diameter of 150 mm. When $k = 1 * 10^{-4}$ and $\Delta H = 4$ m/km, then Q = 52 m^3/h.
In order to carry the peak capacity, a service main should have a higher capacity than the average demand. When a peak hour on a peak day has to be distributed in this main, in case of 100 % connection demand, the total head loss will be 14 m/km. This is too much.
In Mwense the times of supply are irregular. The unreliable water distribution is inconvenient for the consumers.

Conclusions

The base flow of Mwense river, on which the water supply system depends, is too low for the expansion to the gross 100% demand. It is even doubtful if it is possible to abstract the gross demand at the moment.
The capacity of the raw water pumps is not tuned to the demand and the capacity of the treatment and clear water pumps. The capacity of the pump used the months before our visit was much too high, causing pipe bursts in the raw water main and overloading of the treatment, and the pump used later on is too small for the present demand, giving water shortages. Also the capacity of the clear water pump is too small for the gross demand now and at 100% connection of the population. For the capacity it means that the pumps and the alum dosing are the bottlenecks.
When alum is dosed it is not done continues and shortage occur often so there is no proper floc forming and thus the sedimentation has little or no effect, also because the surface load is too high for plain sedimentation.

The filter operation is not very good because the filter is only backwashed when overflowing occurs or dirt is found in the effluent, thus decreasing the effluent quality and wasting water. Backwash rates are too low and the sand has not enough uniformity, causing grading, accumulation of dirt and consequently low filter rates and shorter filter runs.
The mixing conditions for the disinfectant chlorine in the clear are poor causing varying free chlorine concentrations in distributed water.

Recommendations

Alternative sources for raw water should be studied in order to be able to increase the production to the present or future demand.
Proper floc forming is required for good operation of the treatment, that is why it is important to place alum dosing device and to do regular jar test to determine the optimum dosage.
New backwash pipes should make it possible to empty the backwash reservoirs within a few minutes, so backwash rates are increased to produce enough scour for cleaning.
When sand is replaced it is better to use a little bit larger effective grain size(easier to clean) and sieve it for a in order to get a better uniformity.
The chlorine dosing should be placed somewhere where there are good mixing conditions, for example at the inflow of filtered water into the clear water tank.

MWENSE CAPACITY SHEET		
Production now: 20 m^3/h	Gross demand now: 31 m^3/h	Gross 100 % demand: average 43 m^3/h peak day 52 m^3/h peak hour 93 m^3/h
maximum	capacity in m^3/h	100% parameters
raw water pumps	15	347 % of the day
raw water main	52	dH = 4 m/km
sedimentation	32 (plain) 96 (flocculent)	$s_0 = 3.27$
filters	60 (design) 120 (maximum)	q = 8.75 m/h
clear water pumps	24	217 % of the day
clear water main	49	dH = 4.5 m/km
distribution	52	dH = 14 m/km

2.9 NCHELENGE

Introduction

The water supply scheme of Nchelenge embodies an intake from lake Mweru from where the raw water is pumped to the clear water tank. The treatment only consists of disinfection at the clear water tank. From there part of the water is pumped to an elevated service reservoir frome where the distribution of the higher parts of the township takes place by gravity. Part of the water is pumped directly into the reticulation system and part of the water is distributed from the clear water tank (high ground reservoir)

Intake of raw water

Because the raw water is abstracted from a large lake, it is not likely that there will ever be problems with the quantity of raw water. Due to the processes in this lake, fluctuations in the raw water quality are very small.
There is one suction line going into the lake. The intake is too close to the shore where weeds are growing and the depth of the lake is small.
When the water level of the lake drops, the suction line has to be extended, which makes the intake device provisional and vulnerable.
There are no spare pumps. When the raw water pump breaks down there is no water supply anymore. The pump that is used has a capacity of 33 m^3/h, This is not enough for the gross 100 % connection demand.
The raw water main is 75 mm AC/GI. When $k = 1 * 10^{-4}$, $\Delta H = 10$ m and $L = 500$ m, than $Q = 19$ m^3/h. Thus the raw water pump and main are not geared to one another.
All pumps are electrical pumps and power is supplied by Zesco. Especially during the rainy season regular breakdowns occur.

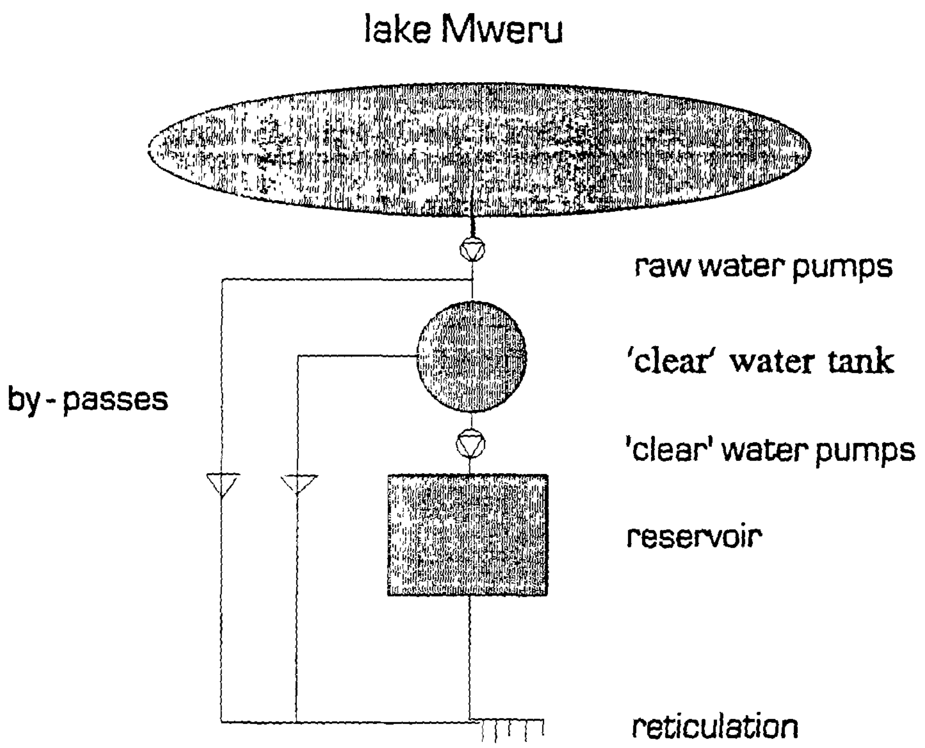
Treatment

There is no water treatment at all.

Disinfection

Part of the water is distributed without disinfection. Since there is no treatment at all, not disinfected water can give serious dangers for the consumers, especially when there is a outbreak of for example cholera or diarrhoea. Part of the water is chlorinated with 1.08 kg of HTH powder, continuously. The average dosage of active chlorine over the whole production is 1.7 p.p.m.. This is not enough. Also mixing conditions are very poor.

Nchelenge



system layout

'Clear' water pump

The capacity of this pump is approximately 3 m^3/h. It is meant to deliver water to the highest parts of town.

Service reservoirs

The total capacity is 124 m^3 which is 14 % of the present daily production and 9 % of the peak day for the gross 100% demand. This is much too small for guaranteed supply.

Distribution

There are two service reservoirs of which the small one is located on a high part of town for supplying the higher areas and which is fed by the 'clear' water pump, and the other larger one is situated on a lower level and is fed by the raw water pump. Distribution is done by gravity from the two reservoirs(each a 50 mm main) and directly from the raw water main(this last part is not disinfected). The service reservoirs are leaking badly, so a lot of water is wasted and the construction of both reservoirs is deteriorating.
The service main from the reservoirs are 50 mm GI. When $k = 1.5 * 10^{-4}$ and $\Delta H =$ 8m/km than Q = 4 m^3/h. The big service main is the raw water main of which the maximum capacity is 19 m^3/h. The total capacity then is 27 m^3/h.
Because raw water is distributed without any treatment, the quality of distributed water is the same as the raw water, causing relatively high turbidities, settling of material in mains and reservoirs and dirt coming out of the taps.

NCHELENGE CAPACITY	SHEET	
Production now: 36 m^3/h	Demand now: 24 m^3/h	100 % Demand: average 49 m^3/h peak day 58 m^3/h peak hour 105 m^3/h
maximum	capacity in m^3/h	100% parameters
raw water pumps	33	176 % of the day
raw water main	19	dH = 19 m/km
distribution	27	dH = 27 m/km

Conclusions

Raw water is distributed what gives unaesthetic situations such as 'mud' coming out of the tap regularly during the rainy season. Because also part of the water is even not disinfected there are serious risks for the consumers, think for example of a cholera outbreak. This situation is unacceptable.
The capacity of pumps and mains is too low for the gross 100% demand.

Recommendations

In the first place all the distributed water must be disinfected. The only place for this is straight after the raw water pumps into the raw water main. The dosage should be 3 to 4 grams of HTH powder per m^3 distributed water.
Because distribution of raw water, and often 'mud', is not acceptable, raw water treatment is required. A study has been executed in cooperation with the World Bank but nothing is yet done with it.

2.10 NYIMBA

Introduction

The water supply scheme of Nyimba embodies an intake from the reservoir at Chikuyu river from where the raw water is pumped to the treatment works. The treatment consists of two regtangular, horizontal flow, sedimentation tanks and two slow sand filters. Disinfection takes place at the clear water tank from where the water is pumped to a large high ground reservoir and a smaller elevated reservoir. From there the distribution to town takes place by gravity.

Intake of raw water

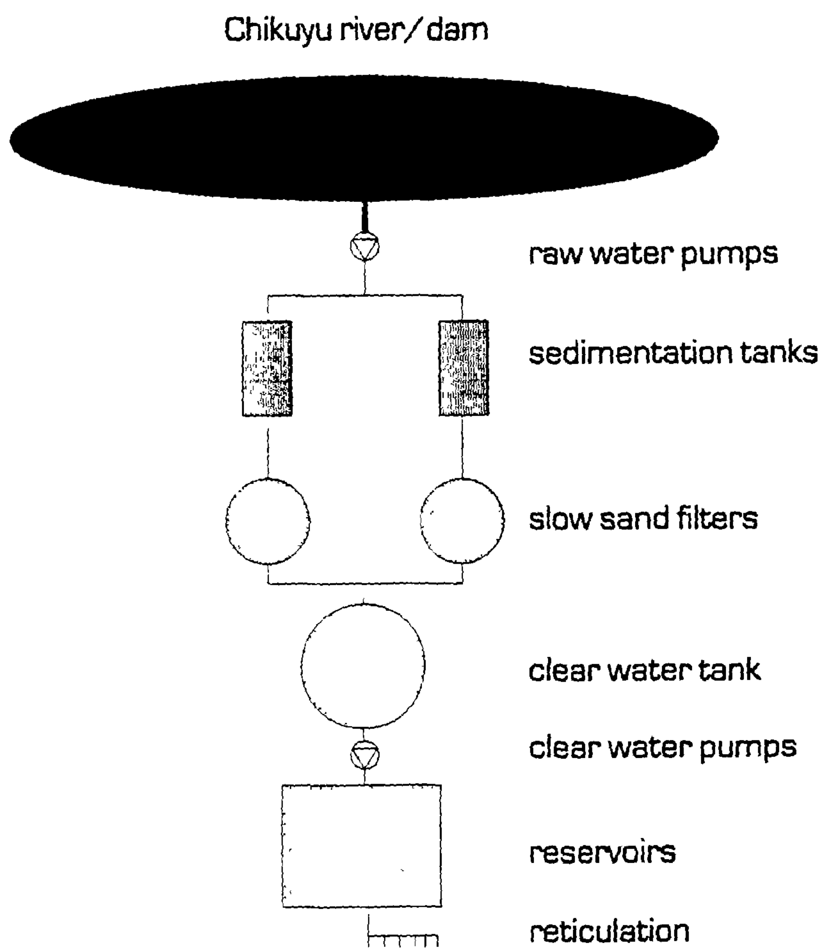
The design capacity of the reservoir is 155600 m^3.
Due to silting problems, the capacity is probable less now. The water production is now 8820 m^3/month which gives a storage of about 12 month when a safety factor 1.5 is taken into account. For the gross 100 % connection demand, 28296 m^3/month, the storage is only 3.6 month. This is not enough to be able to supply water during the whole dry season. A more precise study is to determine the present capacity of the raw water reservoir is required.
Two raw water pumps (one is a stand by pump), $Q = 30$ m^3/h. This is too small for the gross 100 % connection demand.
The raw water main has a diameter of 150 mm. When $k = 1 * 10^{-4}$ and $\Delta H = 4$ m/km, then $Q = 49$ m^3/h This is sufficient for the gross 100 % connection demand.
Power for the electrical pumps is coming from a generator for which regular fuel shortages occur, making water supply impossible.

Treatment

Floc forming and removal

Coagulation
In the rainy season 4 kg of alum is dosed every day. The alum is dissolved in a container to which water is added continuously. So the concentration of the solution decreases during the day.
The rapid mixing takes place where the raw water and the alum solution enter the sedimentation tank. Here the water is very turbulent, so the rapid mixing seems to be good.
When the right amount of alum is dosed at a constant rate and concentration, proper coagulation would be possible. To determine the optimal dosage, jar tests should be carried out regularly.

Nyimba



system layout

Flocculation
There is no flocculation chamber so flocculation simply does not occur. The result is that no larger flocs, which can settle, are formed, thus reducing the effect of the sedimentation.

Sedimentation
There are two sedimentation tanks with an estimated size of 36 m^2 (3*12m) each. The ratio between length and width is 4 which is good for a stable flow. The inlet zone takes care of an equal distribution of water over the width of the tank. The outlet zone is designed wrong because water is discharged only at one point which causes short circuiting and eddies. One of the tanks is leaking badly and can not be used.
With the present improper floc forming only plain sedimentation occurs. The average production 30 m^3/h which gives a surface load of 0.8 m/h when one tank is in use. The maximum surface load of 1 m/h for plain settling gives a production 36 with one tank and 72 m^3/h with both tanks in use. But only relatively course material is removed by plain sedimentation. With proper floc forming flocculent settling will occur which will give better treatment and a higher maximum surface loads. With two tanks in operation the maximum capacity for flocculent settling is 216 m^3/h. For the gross 100 % connection demand this is more than enough.

Filtration

Control
The filters are designed as 'inlet controlled' filters. At time of visit the supernatant level on the filters was very low, due to the way they are operated: the raw water pumps stop pumping at 24.00 hours and from that time till 14.00 hours no water is added to the filters. The filter outlet stays open so the filter keeps running. The result is that the filter rate will eventually drop to zero. The inflow should be constant, which gives a rising supernatant level during operation and constant filter rates.
For slow sand filters the maximum filter rate is 0.25 m/h.
When two filters are in use: 2112 m^3/day , so 63360 m^3/month can be produced. The gross 100 %-connection demand is 28296 m^3/month so the capacity of the filters is sufficient.
The average filter rate now is 0.06 m/h. This is very low and filters should be cleaned before the filter rate drops too low.

Sand
Uniformity U of the used sand was found to be 2.92. This is acceptable for slow sand filters. The d_{10} = 0.59 which is too large for slow sand filtration, and high filter rates but relatively low effluent qualities can be expected.

Cleaning
Before cleaning the filter is dried completely, which destroys the biological layer in the filter bed. As a result the biological treatment will never be optimal because it takes about two month to built up a new biological layer, which is longer than the period within two cleanings.

Disinfection

500 g HTH powder is used every day by adding 100 g in a chlorination device five times a day. Since water is added continuously, the concentration of the solution decreases in between addings. The average concentration of active chlorine is 1.2 p.p.m.
No water should be added in between addings in order to dose with a constant concentration. The mixing conditions are very poor in the clear water tank which causes varying concentrations of free chlorine in the distributed water.

Clear water tank

The tank is in a good state. The capacity of the tank is 454 m^3, this is 154 % of the daily production and 40 % of the peak day 100 %-connection demand. This is enough.

Clear water pumps

Two pumps are there, of which one is a spare pump. When one pump breaks down the other can be used to be able to continue the supply to town.
The capacity of the pump is approximately 30 m^3/h. This means the maximum capacity is 720 m^3/day = 21600 m^3/month. This is enough for the gross 100 % connection demand.

Clear water main

The size of this main is 150 mm AC, the length is 500 m. If $k = 1 * 10^{-4}$ and $\Delta H = 4$ m/km then $Q = 49$ m^3/h. This is enough for the gross 100 % connection demand.

Service reservoirs

The total capacity of the reservoirs is 700 m^3. This is 238 % of the daily production, and 62 % of the peak day for the gross 100% demand, which is enough for all the functions it has.
The large ground reservoir has open windows what allows contamination of the treated water by animals.

Distribution

Higher parts of town only get water 3 hours a day. This is very inconvenient for the consumers. There are said to be no serious leakages, which is doubtful.
No spare parts are available. When a main gets broken and has to be replaced it will take some time to repair it.
There are three big service mains going out of the reservoirs, 80 mm each. When $k = 1 * 10^{-4}$ and $\Delta H = 4$ m/km, than $Q = 10$ m^3/h for each main, which is much to small for the gross 100% demand.

NYIMBA CAPACITY	SHEET	
Production now: 12 m^3/h	Demand now: 26 m^3/h	100 % Demand: average 39 m^3/h peak day 47 m^3/h peak hour 84 m^3/h
maximum	capacity in m^3/h	100% parameters
raw water pumps	30	157 % of the day
raw water main	49	dH = 3.6 m/km
sedimentation	72	s_0=0.65
filters	88	q =0.13 m/h
clear water pumps	30	157 % of the day
clear water main	49	dH = 3.6 m/km
distribution	30	dH = 35 m/km

Conclusions

The capacity of the pumps and the distribution system is too small to handle the gross 100% demand. Also the capacity of the raw water reservoir is too small for supply during the whole dry season if the capacity of the system is raised to the gross 100% demand.
Proper floc forming is not possible because there is no flocculation chamber.
Consequently only plain settling will occur in the sedimentation tank, removing only relatively coarse material. That is why high turbidity loads on the slow sand filters are expected during the rainy season, shortening filter runs and decreasing filter rates.
Because the raw water pump is not continuously working but the filters are, the filter rate is constantly varying, causing more or less declining rate filtration instead of constant rate and making it hard to determine how far a filter is clogged .
The cleaning procedure causes deterioration of the biological layer while with every cleaning the filter bed is dried completely. Consequently the biological clarification is not very good.
The effective diameter of the sand is too large.
The chlorine dosage is a too low and mixing conditions are poor in the clear water tank.

Recommendations

A study should be carried out in order to find the exact storage capacity of the raw water reservoir, which is probably too small for the gross 100% demand. Also alternative sources should be found.

Raw water pumps should be working more regular in order to get a more continues operation (and better filter control) of the treatment.

In order to get proper floc forming and removal, and thus lower turbidity loads on the slow sand filters, a flocculation chamber should be placed before the sedimentation. Coagulant dosage should be done more precise and regular jar tests should be carried out to determine the optimal dosage of alum.

The biological layer can be kept intact by not drying the filter bed completely during cleanings, but just draining it to about 0.2 metres under the surface of the bed as is necessary for manual cleaning.
The cleaning procedure than changes to the following:

a. drain the filter to approx. 0.2 metres beneath the sand surface
b. directly after this scrape off the upper 2 to 3 cm of the sand bed
c. refill the filter again by backfilling.

The present sand should be replaced by sand with a smaller effective size, d_{10} between 0.1 and 0.3 mm.

2.11 SAMFYA

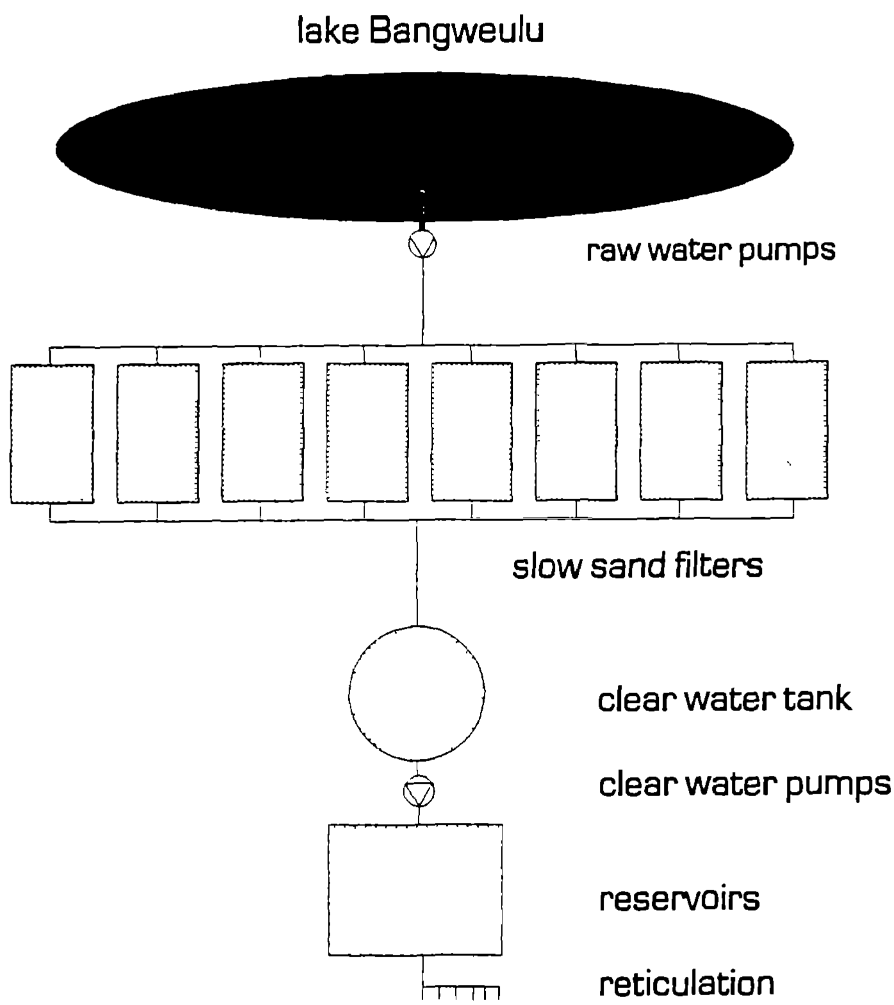
Introduction

The water supply scheme of Samfya embodies an intake from lake Banguelu from where the raw water is pumped to the treatment works. The treatment consists of eigth slow sand filters. Disinfection takes place at the clear water tank from where the water is pumped to the high ground service reservoirs. From there the distribution to town takes place by gravity. A small amount of water is pumped from these reservoirs to an elevated reservoir for supply of the higher parts of the township.

Intake of raw water

Water is abstracted from lake Bangweulu which water level is even during very dry years high enough for good abstraction.
During the rainy season the raw water is said to be very turbid which can cause a too rapid clogging of the slow sand filters. To verify this regular turbidity measurements over a period of at least one year should be carried out.
The intake device is placed and protected well.
There are 3 raw water pumps each with a given capacity of 90 m^3/h at a head of 8 metres and it is possible to operate two pumps at the same time. Measurements showed an inflow of raw water of only 76 m^3/h for 1 raw water pump with 4 of the 8 slow sand filters in use. With 2 raw water pumps working at the same time maximum capacity with all 8 filters working will be approx. 150 m^3/h. The head loss over the raw water main to the filters, length 300 m and diameter 200 mm GI, will be about 2,5 metres at a flow rate of 150 m^3/h.
Power for the pumps comes from Zesco and especially during the rainy season regular breakdowns occur.

Samfya



system layout

Filtration

In Samfya 8 slow sand filters have been built in 1986. The raw water from lake Bangweulu is pumped on the filters without any pre-treatment. Year round turbidity measurements of the raw water are not available but should be carried out in order to verify if the turbidity during the rainy season is low enough for direct slow sand filtration.

Control

Negative pressure will not occur while the pressure in the effluent pipe is always around 20 cm above the surface of the sand bed.
The filters have been built as outlet controlled. The effluent pipe of each filter contains a butterfly valve with which the head loss over the outlet can be controlled. This valve should be opened bit by bit as the filter clogs during operation.
The present situation is that the filters are used inlet controlled. The outlet valve is during operation completely opened and the operators the inlet for raw water to control the supernatant level. This means that all the filters are kept at more or less the same supernatant level, thus clean filters take a lot of water, filter rates higher than 0.4 m/h, and clogged filters just a little bit, filter rates less than 0.1 m/h. In this way filters clog more rapidly.
Filter runs are about 7 to 10 days, so almost every week filters have to be cleaned. In order to higher the filter rates and lengthen the filter runs, 6 to 10 cylinders with a diameter of 150 mm filled with gravel were placed as a sort of vertical drains in every filter bed. Consequently a lot of water by-passes the filter bed thus reducing the effluent quality for both turbidity and micro organisms. Operators told the filter runs exceeded then to 14 days during the rainy season and a month during the dry season. While the filter runs during the rainy season are twice as short as during the dry season too high turbidity loads on the filters are expected during the rainy season.
Often supernatant water is lost through the overflow device while the total inflow into the filters exceeds the capacity of the used filters.

During our visit the cause was that only 4 out of 8 filters were in use, with an average filter rate of 0.15 m/h the total discharge is 65 m^3/h while the inflow was approx. 75 m^3/h. Thus 15 % of the energy costs of the raw water pumps is wasted.
The maximum average filter rate is approx. 0.25 m/h which gives maximum production for the filters of 216 m^3/h
The effective size d_{10} of the sand is 0.35 mm which is fair-sized. The coefficient for uniformity, U = 1.86, is good.

Two basic mistakes are made with the cleaning procedure. In the first place, filters are drained and dried completely for every cleaning session thus reducing strongly the clarification effect of the biological layer. Second, cleaning is done by scraping off only the very thin dried out dirt layer on top of the sand bed, whereas it is necessary to remove the upper five centimetres of the sand bed. Now dirt can accumulate in the upper layer of the sand bed causing extra resistance against flow, thus decreasing the available head loss and filter runs.

There was one strange thing, the top layer of the dried sand beds was very hard, like stone. Lab analysis(at the end of this analysis) on this top layer showed very low concentrations organic, suspended and collidal matter. Also there was no visually perceptible reaction with HCL, so almost no carbonates were present in the sand.
Still, probalbly the cleaning procedure is one of the causes of the problem but it is not sure if it is the only one. As the used pH meter was inaccurate it was impossible to calculate if the water is under or supersaturated (Calcium-carbonic acid equilibrium). Also it is unknown how the algae concentration varies over the year. These last two are important for the full perception of the problem and causal relations.

Disinfection

Disinfectant is dosed directly and continuously into the clear water tank where proper mixing is absent. The contact time, slightly more than one hour in the clear water tank only, is long enough.
The total dosage is not enough, 1.4 kg HTH a day with a production of 1550 m^3/day gives an average concentration of 0.7 mg/l or p.p.m.

Clear water tank

The capacity of the clear water tank is 82 m^3 which is 5 % of the daily production and 2.5 % of the peak day for the gross 100 demand, which is too small. Overflowing or running dry of the clear water tank is reported regularly. The main reason for this is that the total discharge of the filters to the clear water tank is constantly changing while there is declining rate filtration. Another reason is that the clear water pumps have a smaller capacity than the raw water pumps.

Clear water pumps

There are two clear water pumps each with a capacity of 70 m^3/h and of which one presumably is meant as a spare pump. Compared with three raw water pumps each with a capacity of 90 m^3/h this is very strange. At the time of visit the clear water pumps formed the limiting factor in the production. Only four filters were in operation and one clear water pump was working 24 hours a day. A possibility is to use the two clear water pumps at the same time, but then first the water hammer provision must be repaired.

Clear water main

The water hammer provision is not working at the moment while the compressor which is

used to maintain the right pressure in two pressure vessels is broken.
The capacity of the main, 200mm GI with a length of 500 m, is 150 m^3/h if a head loss due to friction of 4 metres is allowed.

Service reservoirs

The total reservoir capacity is sufficient, 1670 m^3 which is 107 % of the daily production and 57 % of the peak day for the gross 100% demand. This is sufficient.

Distribution

Many leakages have been reported and due to lack of spares the repairing is not optimal. Only 27 % of the population in Samfya is officially connected to the distribution system so a lot of illegal use of water is expected.
The two big service mains, 150 and 200 mm, have a total capacity of 155 m^3/h if calculated with a dH = 4 m/km.

SAMFYA CAPACITY SHEET		
Production now: 65 m^3/h	Gross demand now: 50 m^3/h	Gross 100% demand: average 103 m^3/h peak day 123 m^3/h peak hour 222 m^3/h
maximum	capacity in m^3/h	100% parameters
raw water pumps	150	82 % of the day
raw water main	150	dH = 5.5 m/km
sedimentation	-	-
filters	216	q = 0.14 m/h
clear water pumps	70	176 % of the day
clear water main	150	dH = 5.5 m/km
distribution	155	dH = 8 m/km

Conclusions

The filter runs are too short, about 7 to 10 days only. The exact cause of this is still not fully known, but one of the reason is the cleaning procedure which allows organic matter and small colloidal particles to accumulate in the uppermost centimetres of the sand bed, therefore causing extra resistance against flow. The decreased available head loss shortens the filter runs. Algae problems are reported but are not alwyas there, and the Calcium-carbonic acid equilibrium is not known.
Also the cleaning procedure causes deterioration of the biological layer while with every cleaning the filter bed is dried completely. Consequently the biological clarification is not very good.
The filter control is not as it is designed, causing declining rate filtration.
The sets of raw and clear water pumps are not geared to one another. One extra clear water pump of the same type as spare and repairing of the water hammer device makes it possible to operate with two clear water pumps at the same time, bringing the capacity at approx. 130 m^3/h, which is enough for the gross demand when 100% of the population is connected to the water supply system. The needed head loss in the distribution lines with all the population connected is probably not available so the capacity has to be extended when the supply grows to the gross 100% demand. The capacity of the other components is large enough for the gross 100% demand.

Recommendations

Carry out year round turbidity and algae measurements on the raw water in order to determine if the turbidity during the whole year is low enough for direct slow sand filtration and how the algae concentration varies and also if can cause serious problems for filter operation. Also the Calcium-carbonic acid equilibrium should be checked in order to see if the water is supersaturated (which means: able to deposit Calcium-carbonate) or not.

The accumulation of dirt in the top layer of the sand bed can be prevented by scraping off the upper five centimetres of the sand bed with every cleaning. The complete sand bed is then renewed within about two years.
Best to start with now is to renew all the sand, or at least renew the upper half of the sand bed. In the last case, the bottom half of the sand bed that is reused has to be removed and than put on top of the new sand. The new sand is best to have the same characteristics as the sand that is used at the moment.
The biological layer can be kept intact by not drying the filter bed completely during cleanings, but just draining it to about 0.2 metres under the surface of the bed as is necessary for manual cleaning.
The cleaning procedure than changes to the following:

a. drain the filter to approx. 0.2 metres beneath the sand surface
b. directly after this scrape off the upper 2 to 3 cm of the sand

c. refill the filter again by backfilling.

In order to improve the filter control, the head loss over the effluent outlet valve of each filter should be gradually downgraded proportional to the increasing bed resistance during operation. The inlet valves must be opened completely to get an equal distribution of raw water over all the working filters.

Another, easier option is to operate the filters with a rising supernatant level. Again the inlet valves need to be opened completely for an equal distribution of raw water but also the outlet valves can remain opened completely now without adjusting. Clean filters will have a relatively low supernatant level which will rise as the filter clogs. A filter should be cleaned as the supernatant level reaches the overflow level. The overflow weir for filtered water flowing into the clear water tank prevents the supernatant level to drop further than about 20 cm above the sand bed surface, thus it prevents negative pressure and running dry of the filter.

Place a third clear water pump of the same type and repair the compressor for the water hammer provision. In this way pumping with two clear water pumps at the same time is possible thus increasing the overall capacity to approx. 130 m^3/h (this is still with the present average filter rate of 0.15 m/h and 8 filters in operation), which is more than enough for the present demand in Samfya.

2.12 SIAVONGA

Introduction

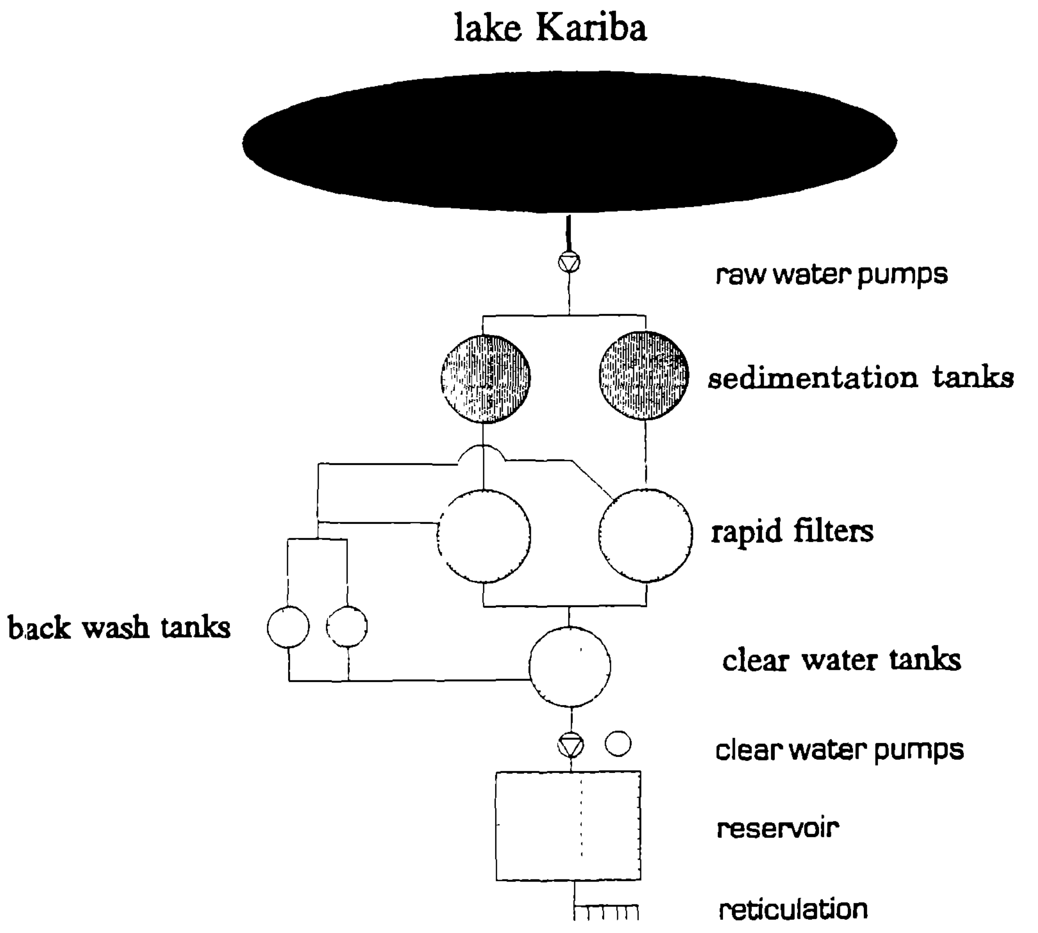
The water supply scheme of Siavonga embodies an intake from lake Kariba from where the raw water is pumped to the treatment works. The treatment consists of two circular, upflow combined floc forming/sedimntation tanks that work with a sludge blanket, two rapid filters and two back wash tanks. Disinfection takes place in the clear water tank(ground reservoir at a high point) from where the water distributed to town by gravity.

Intake of raw water

Raw water is abstracted from the large lake Kariba so quantity problems are not expected.

The intake construction is provisional. Due to the low level of lake Kariba the last few years, intake mains are extended and extra pumps are placed just on the with stones covered lake shore in order to pump the raw water up to the original pumps. The construction is very vulnerable. The suction line is not going very deep into the lake so weeds can block the suction head.
There are spare pumps so in case of a breakdown of a pump another pump can be started and the treatment process can be continued.
There are two raw water pumps, $Q = 75\ m^3/h$ and $55\ m^3/h$; $H = 90$ m
The raw water mains are 100 mm GI and 150 mm AC, $\Delta H_s = 75$ m.
When $k = 1.5 * 10^{-4}$, $d = 100$ mm, $\Delta H = 8$ m/km and $L = 300$ m than $Q = 25\ m^3/h$.
When $k = 1 * 10^{-4}$, $d = 150$ mm, $H = 8$ m/km and $L = 300$ m, than $Q = 75\ m^3/h$.
These last two capacities are calculated for a head loss due to resistance of 8 m/km, but the available head loss is higher because the pumps can handle about 90 meters of total head while the static head is only 75 metres.

Siavonga



system layout

In this case, about 55 m^3/h is pumped through the 100 mm main at a total head of about 90 metres and about 80 m^3/h is pumped through the 150 mm main at more or less the same head.
Power for the pumps comes from the power plant at the Kariba dam, the supply is very good.

Treatment

Floc forming and removal

Normally a daily amount of 1 kg of alum is dosed into only one of the two sedimentation tanks. The second tank never gets alum. At time of the visit there had not been alum for nine month. When the turbidity of the raw water is low enough for direct filtration this is no real problem, but in Siavonga the rapid filters a said to be backwashed six times a day during the rainy season, so proper pre treatment is certainly required.
The right amount of coagulant should be dosed continuously with a constant concentration in order to get good coagulation. Also good rapid mixing of coagulant with the raw water is important. The dosage should be determined by executing a jar test every day or week (depending on the variation of the turbidity). When operators in charge have a lot of experience, dosing on sight can be sufficient.

The two sedimentation tanks are of the circular 'sludge blanket' type as explained in the chapter 'theory' of this appendix. With surface loads between 1 and 3 m/h, Q between 16 and 48 m^3/h (for flocculent settling which only occurs with proper coagulant dosing), the rapid mixing down in the tank and the flocculation in the cone are sufficient.
At the plant operators expect the sludge to settle at the bottom where no drain can be found and at times empty the tank manually to remove the sludge.
When no alum is dosed the tank is used as a plain sedimentation tank, which only removes relatively course material. With a surface area $A = 15.9\ m^2$ and an average production of 63 m^3/h, the surface load $s_0 = 2.0$ m/h for each tank, which is much too high for plain sedimentation and makes the whole sedimentation not very effective.
With the larger raw water pump working the surface load is even higher. For plain sedimentation the maximum surface load $s_0 = 1$, and the maximum flow rate is found: $Q = 15.9\ m^3/h$ for each tank, totalling 32 m^3/h.

Filtration

Control
An overflow weir in the effluent line to the clear water tank protects the rapid filters against negative pressures. This is very useful for good operation. Filter control is done on sight: when the water level rises too high the backwash procedure is started.
Backwashing is done 4 times a day in the dry season and 6 times a day in the rainy season, this is more or less a standard procedure. The backwash is stopped on sight,

when the wash water looks clean again.
The system is designed to be inlet controlled with a rising supernatant level and is treated like that. Only far too many backwashing are carried out, wasting a lot of time and water. should start before the filters overflow and water is wasted.
The design filter rate is 5 m/h while the average filter rate is 5.25 m/h. A maximum average filter rate of 10 m/h, gives a maximum production of 120 m^3/h. The design rate gives a capacity of 60 m_3/h (total for two filters).

Sand
The uniformity of the sand, U = 1.47, is ok. The effective size of the sand grains d_{10} = 0.75 mm is pretty small which makes it hard to produce enough scour with backwashing in order to keep the sand clean.

Backwash
Two backwash tanks of 9 m^3 each are built on top of the office, 6 to 8 m above the water level in the filter. For backwashing of a filter both tanks are only used till the wash water looks clean. While the filter bed area is 6 m^2 a backwash reservoir with a capacity between 18 and 36 m^3 placed at least 4 m above the water level in the filter is required. Thus the present backwash facilities are not enough because pretty small sand is used which requires high backwash rates. The sand used, with d_{50} = 1.05 mm, requires a backwash rate of about 60 m^3/m^2/h (T = 20°C), or 360 m^3/h for the whole filter, in order to get enough expansion and scour to keep the sand bed clean. The two backwash reservoirs thus need to be emptied each within 90 seconds making the total backwash time 180 seconds. The present situation is not like that, backwashing takes about 15 minutes so backwash rates are too low. This results in accumulation of dirt, filter cracks and mud balls and consequently lower filter rates and shorter filter runs. This is one of the reasons for the abnormal number of backwashes. Other reasons are the high turbidity loads on the filters as pre treatment without coagulant dosing has little or no effect and the standard procedure that a filter has to be backwashed four or six times a day, even when it is probably not necessary. At time of visit the raw water turbidity was only 1.2 NTU and after filtration it was 0.4, and still they kept on doing all these backwashes.

Disinfection

Every 6 hours, 600 g HTH powder is dosed. This is 1.4 p.p.m. what is not sufficient. The dosing takes place at the outlet of one of the filters. In the line of that filter the chlorine is mixed good. The mixing with the water from the other filter has to take place in the clear water tanks where mixing conditions are poor, so not all produced water is disinfected very good. Better is dosing HTH in both filter effluent outlets.
The contact time is good, several hours in the clear water tank.

Clear water tanks

The tanks have a total capacity of 990 m^3. This is 65 % of the daily production and the

same for the peak day at with the gross 100% connection demand. This is enough for the storage of differences between production and demand over the day and is even reasonable storage for supply in case of emergencies (fire) and system breakdowns.
The clear water tanks are located at such a height, that no elevated reservoirs are needed.

Distribution

One 100 mm main is coming from the clear water tanks. Further on this main is divided into 4 service mains of the same size, 100 mm each. When $k = 1 * 10^{-4}$, $H = 75$ m and $l = 3000$ m, than the capacity of this main is $Q = 50\ m^3/h$ because the available head loss from the clear water reservoir to the shores of lake Kariba is about 75 metres. In reality many parts of town do not get any water because the distribution system is not one straight line down to the lake. So in the first place the pipes are too small and second, a pipe should split in two pipes with a diameter smaller than its own as otherwise there will always be a shortage in these last two pipes.

SIAVONGA CAPACITY	SHEET	
Production now: 63 m^3/h	Demand now : 50 m^3/h	100 % Demand: average 52 m^3/h peak day 62 m^3/h peak hour 112 m^3/h
maximum	capacity in m^3/h	100% parameters
raw water pumps	75 + 55	
raw water mains	75 + 25	dH = 5.8 m/km
sedimentation	31.8 (plain) 95 (alum dosed)	s_0= 1.95
filters	60(design) 120(maximum)	q = 5.2 m/h
distribution	50	dH = 16 m/km

Conclusions

The raw water intake structure is provisional and vulnerable. The main reason for this is the low water level of lake Kariba which forced the intake device to be moved downwards and made booster pumps necessary.
Because no coagulant has been dosed for nine month (at time of visit), the sedimentation has almost no effect on the water quality and thus the system works with more or less

direct filtration of raw water.
Far too many backwashes are carried out every day, about 6 in rainy season and 4 in the dry season. A lot of water is wasted here. Main reasons for all these backwashes are:
* It is a standard procedure to do all these backwashes, so even in case it is not necessary operators keep on backwashing.
* Due to high turbidity loads, because of direct filtration, during the rainy season.
* Backwash rates are too low so dirt can accumulate in the sand and shorten filter runs.

As part of the filtered water is not disinfected thoroughly pathogenic organisms can still be alive in the distributed water.

Recommendations

Most important is to reduce the number of backwashes. First, a filter should only be backwashed if the supernatant level is really at its maximum level. Second, a coagulant (alum) should be dosed if the turbidity of the raw water is too high, this is when filter runs are shorter than about 24 hours, in order to make the pre treatment effective. Frequent jar tests should be carried out to determine the most effective alum dosage. Third, the backwash rates should be increased which can be done increasing the size of the pipes in such a way that emptying the two backwash reservoirs takes only a few minutes.
Proper disinfection can be obtained by placing also a dosing device at the outlet of the filter that has not one at the moment and dose about 3 to 4 grams of HTH per m^3 produced water.

2.13 ZIMBA

Introduction

The water supply scheme of Zimba embodies an intake from the two reservoirs at Zimba and Railway dam from where the raw water is pumped to the treatment works. The treatment consists of a baffle channel, one regtangular, horizontal flow, sedimentation tank and two slow sand filters. Disinfection takes place in the clear water tank from where the water is pumped to the elevated service reservoirs. From there the distribution to town takes place by gravity.

Intake of raw water

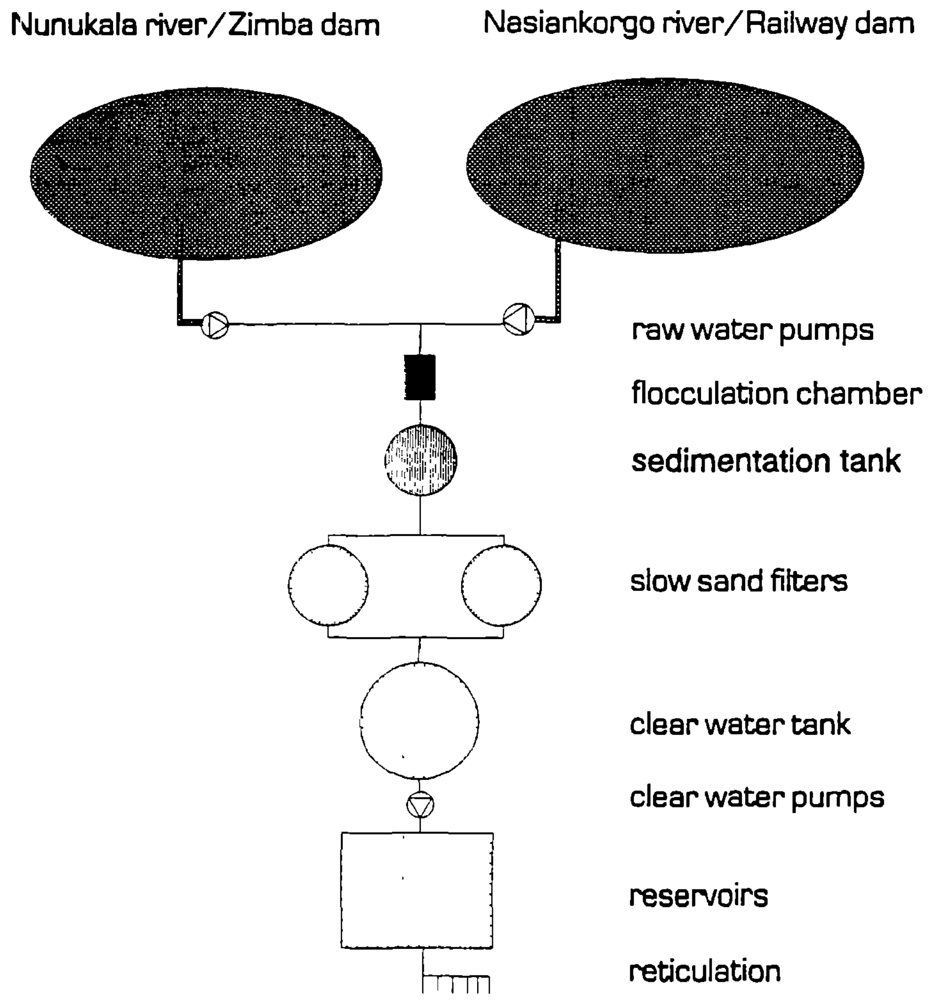
The total capacity of the two raw water reservoirs is 468780 m^3
The gross demand when 100 % of population is connected is 31862 m^3/month. In the dry season the water supply mainly relays on the capacity of the reservoirs since nearly no water is added by the river.
This means that there has to be enough storage in the reservoirs to last for 7 months: 223030 m^3. This is 48 % of the total capacity of the reservoirs. So although the effective storage of the reservoirs probably is less than the design capacity due to silting and evaporation, there still will be enough to serve the town.
Erosion of cultivated land in the river basin is said be the cause of high turbidities of the raw water in the rainy season and silting in the reservoirs.

Power for the pumps comes from Zesco. Especially during the rainy season regular breakdowns occur.

Zimba dam
Due to seepage the contents of the reservoir reduces with time. Seepage can cause piping which can result in undermining of the dam. The spillway is deteriorating due to lack of maintenance.

The raw water main is led through the spillway. When high reservoir levels rains occur and the spillway discharges a lot of water, the raw water main might get damaged. Silting problems are said to reduce the capacity of the reservoir.
The intake normally takes place 5.85 m below the water level. The system that keeps the intake line at the right level has sunk so now the intake takes place at the bottom of the reservoir. Dirt might be abstracted with raw water, mud can cover the intake device and it is hard to do any maintenance on the intake device.
At this dam there is only one raw water pump, so no standby pumps are available in case of a breakdown. The capacity of the pump is approximately 50 m^3/h.
There is no bulk meter thus the amount of abstracted water is not known.

Zimba



system layout

Railway dam
The long spillway of the railway dam is deteriorated at the end. If this continuous danger of undermining of the structure occurs. The intake is rehabilitated completely in 1985 and seems to work very well.
All air valves of the raw water main are regularly checked and all working so no air pockets are expected in this main.
No flow measurements are done at this intake: there is a bulk meter but the person in charge is not able to read it. This does not make sense.
There are two intake pumps, both are working. One is a standby pump. The capacity of both pumps is 45 m^3/h.

The maximum capacity of the total intake is 2280 m^3/day, this is 68400 $m^3/$ month. The gross 100 % connection demand is 31862 $m^3/month$, so the capacity of the intake pumps is sufficient.

Both raw water mains are 150 mm GS. If $k = 1.5 * 10^{-4}$ and $\Delta H = 4$ m/km then $Q =$ 49 m^3/h for both mains, which is good compared with the pumps and the gross 100% connection demand.

Treatment

Floc forming and removal

Coagulation
Alum is dosed the whole year, but more in rainy season than in dry season. At time of visit there was only for one day of alum in stock. When one runs out of alum, one should report this on time to the provincial water engineer. In this case it was reported too late so there might be no alum for days or weeks. This means no coagulation can take place and rapid clogging of the filters will occur.
The alum dosing system gives a constant solution during the day and adds this solution continuously in the baffle channel.
In dry season 4 kg is added per day, in rainy season 10 kg.
Jar tests, to find the right amount of alum for good coagulation, are never done. Also varying the amount of alum on visual base is not done.
Rapid mixing takes place at the entrance of the baffle channel where alum is added to the inflowing raw water. A lot of turbulence here provides thorough mixing.When the right amount of alum is dosed, proper coagulation is possible.

Flocculation
Flocculation takes place in a baffle channel and is done properly.

Sedimentation

The system has one sedimentation tank, circular, with a diameter of 12 m. With a surface area of 113 m^2 the average production of 20.5 m^3/h gives a surface load of 0.18 m/h. The gross 100% demand 44 m^3/h gives a surface load of 0.39 m/h. Both surface loads are low and therefore effective sedimentation is possible. The maximum capacities are: 113 m^3/h for plain settling and 339 m^3/h for flocculent settling.

Sludge is removed by a washout. After sedimentation a turbidity of 17.9 NTU was found while the raw water turbidity was 42.4 NTU. The removal ratio than is 58%. More effective sedimentation probably possible if the alum dosage is determined with a jar test.

Filtration

Control

The slow sand filters are designed as outlet controlled. Only the resistance over the outlet valve is never adjusted but always at its minimum, so basically there is no control carried out. The result is declining rate filtration instead of constant rate filtration. Filter runs will be shorter and especially at the beginning of a filter run the filter rate will be high, with lower effluent qualities because of the short retention time, and there will be a rapid clogging of the filter bed.

The maximum capacity of the filters, with an average filter rate of 0.25 m/h, is 55 m^3/h.

Sand

Uniformity U is a little bit too high for slow sand filters (U = 3.5). The effective size is good, d_{10} = 0.3 mm. The sand layer is only 15 to 20 cm which is much too thin for proper filtration. No effective biological treatment will be possible, there will be a very short retention time of water in the sand bed and consequently the effluent quality will not be very good. At time of visit the turbidity after filtration was 8.16 NTU while before filtration it was 17.9 NTU, this is not very good.

Cleaning

Before cleaning the filter is dried completely, which destroys the biological layer in the filter bed. As a result the biological treatment will never be optimal because it takes about two month to built up a new biological layer, which is longer than the period within two cleanings.

Disinfection

2 kg of chlorine is added to the clear water tank in the morning, at once, because the dosing device is broken. This results in a high concentration in the morning and almost no chlorine at the end of the day. The average dosage of active chlorine is 2.85 p.p.m. which is sufficient. Mixing conditions in the clear water tank are very poor which also causes varying concentrations.

Clear water tank

The capacity of the clear water tank is 17 % of daily production now and 7.5 % of the peak day with the gross 100 % connection demand. This is just enough.

Clear water pumps

At moment of visit only one pump was working, no stand by pumps.
The capacity of the pump is 23.2 m^3/h. This is just enough for the gross demand at the moment but certainly not for the gross 100% demand.

Clear water main

The main has a diameter of 100 mm.
When $k = 1.5 * 10^{-4}$ and $\Delta H = 4$ m/km, than $Q = 24\ m^3/h$.
When the gross 100 % connection demand has to be transported this main is too small.

Service reservoirs

The total capacity of the service reservoirs is 662 m^3 which is 135% of the daily production and 53% of the daily gross 100% demand. This is large enough to store the differences between production and demand over the day, for fire fighting and even for a few hours supply during breakdowns.

Distribution

The connection percentage is only 27 % which is very low. That is why a lot of illegal connections and draw off can be expected.
Distribution starts with in total three lines coming from the reservoirs.
One 150 mm GI main, when $k = 1.5 * 10^{-4}$ and $\Delta H = 4$ m/km, than the capacity is 49 m^3/h.
One 75 mm AC main, when $k = 1.0 * 10^{-4}$ and $\Delta H = 4$ m/km, than the capacity is 8 m^3/h.
One 50 mm AC main; when $k = 1.0 * 10^{-4}$ and $\Delta H = 4$ m/km, than the capacity is 3 m^3/h.
The total capacity is therefore 60 m^3/h which is enough for the average gross 100% demand, but not for the peak hour.

There is water supply between 6.00-9.00 h, between 12.00-14.00 h and between 17.00 and 22.00 h. At night there is no water supply. This is good for the rationing since there are many leaking taps that would cause considerable losses during the night when there is pressure. But due to this, the mains will corrode easily since air can come in at night and quality of water that is still in the system will deteriorate.

ZIMBA CAPACITY	SHEET	
Production now: 20 m³/h	Demand now: 24 m³/h	100 % Demand: average 44 m³/h peak day 53 m³/h peak hour 95 m³/h
maximum	capacity in m³/h	100% parameters
raw water pumps	50 + 45	56 % of the day
raw water main	49	dH = 4.6 m/km
sedimentation	113 (plain) 339 (flocculent)	s_0= 0.47
filters	56.5	q = 0.23 m/h
clear water pumps	23.2	228 % of the day
clear water main	24	dH = 39 m/km
distribution	60	dH = 10 m/km

Conclusions

Except for the clear water pumps and main, all the components have a capacity large enough for the gross 100% demand.
The intake device at the Zimba Dam reservoir is broken which causes problems with the intake of water, especially the water quality because a lot of mud is abstracted with the raw water.
The pre treatment has been built very well, a good coagulant dosing system, rapid mixing, flocculation and sedimentation. The only problem is the coagulant dosage. Often shortages of alum occur and no jar tests are carried out to determine the right alum dosage. Consequently the pre treatment will still not work optimal.
The filters should be outlet controlled but in practise are not controlled at all(the only control is to stop the pumps if a filter is overflowing), which causes declining rate filtration. Very bad is the sand layer, which is only 15 to 20 cm thick. Therefore no effective biological treatment will be possible, there will be a very short retention time of water in the sand bed and consequently the effluent quality will not be very good.
Before cleaning the filter is dried completely, which destroys the biological layer in the filter bed. As a result the biological treatment will never be optimal because it takes about two month to built up a new biological layer, which is longer than the period within two cleanings.

There is no dosing device for the disinfectant so proper disinfection of all produced water is not possible.
The percentage of the population that is connected to the water supply system is very low in Zimba, about 27% only. Consequently a lot of people have to look for alternative sources for water or take water from the system illegally.

Recommendations

In order to abstract better water from the Zimba Dam reservoir, the intake device should be repaired.
Optimal pre treatment, what can reduce the turbidity on the slow sand filters enormously, is only possible if alum is dosed. Enough alum in stock and early ordering should secure constant dosing.
Frequent jar test should be carried out in order to determine the best dosage.

For effective filtration a sand layer of about 1 meter is required, so the present filters should be filled to that with new sand. The filter control can be done by reducing the resistance over the effluent line in such a way that the discharge is kept during a filter run. If all resistance used, a filter needs to be cleaned.
The biological layer can be kept intact by not drying the filter bed completely during cleanings, but just draining it to about 0.2 metres under the surface of the bed as is necessary for manual cleaning.
The cleaning procedure than changes to the following:

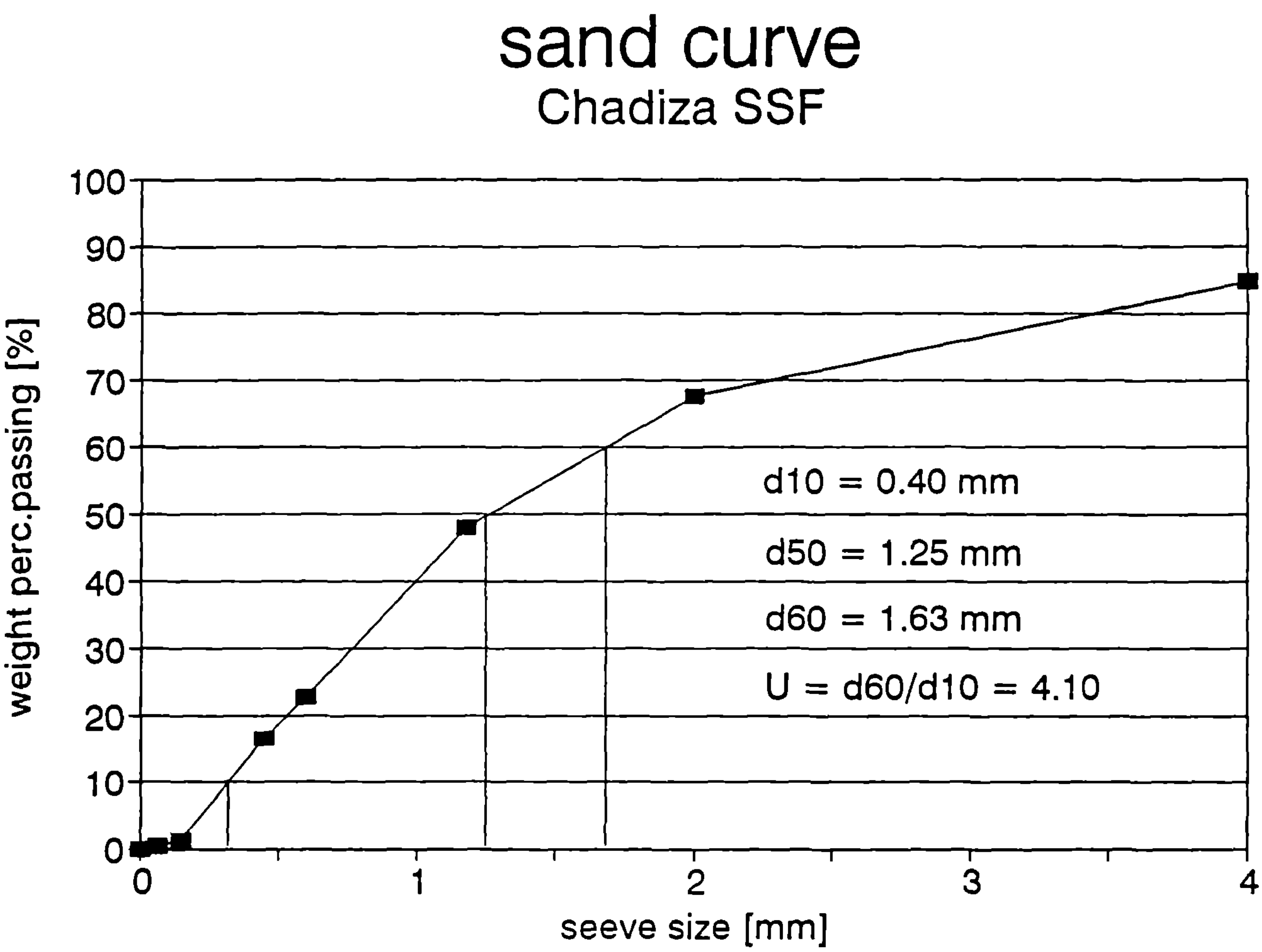
a. drain the filter to approx. 0.2 metres beneath the sand surface
b. directly after this scrape off the upper 2 to 3 cm of the sand
c. refill the filter again by backfilling.

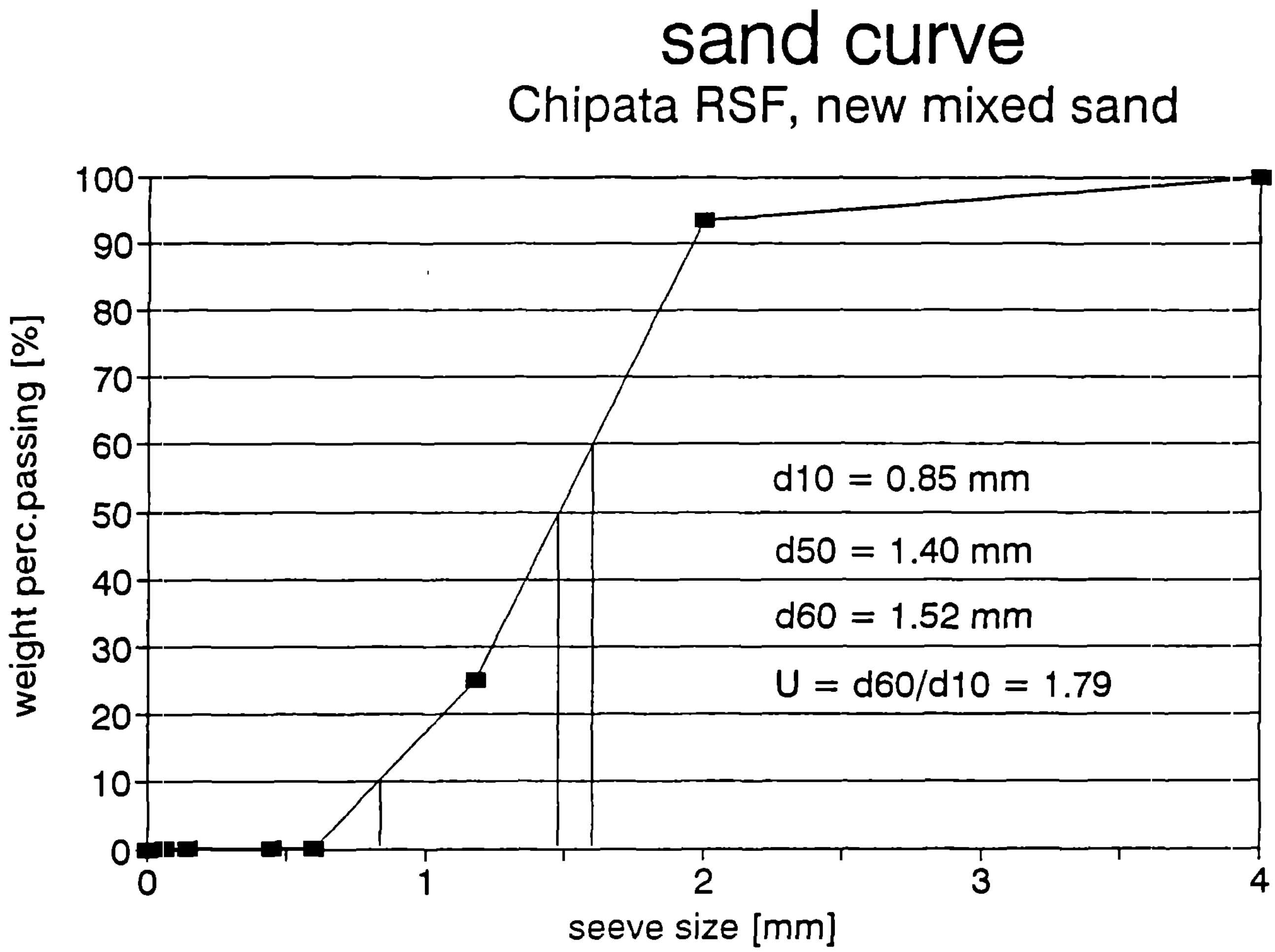
For good disinfection dosing device is required. The dosage of HTH should be 3 to 4 grams per m^3 produced water.

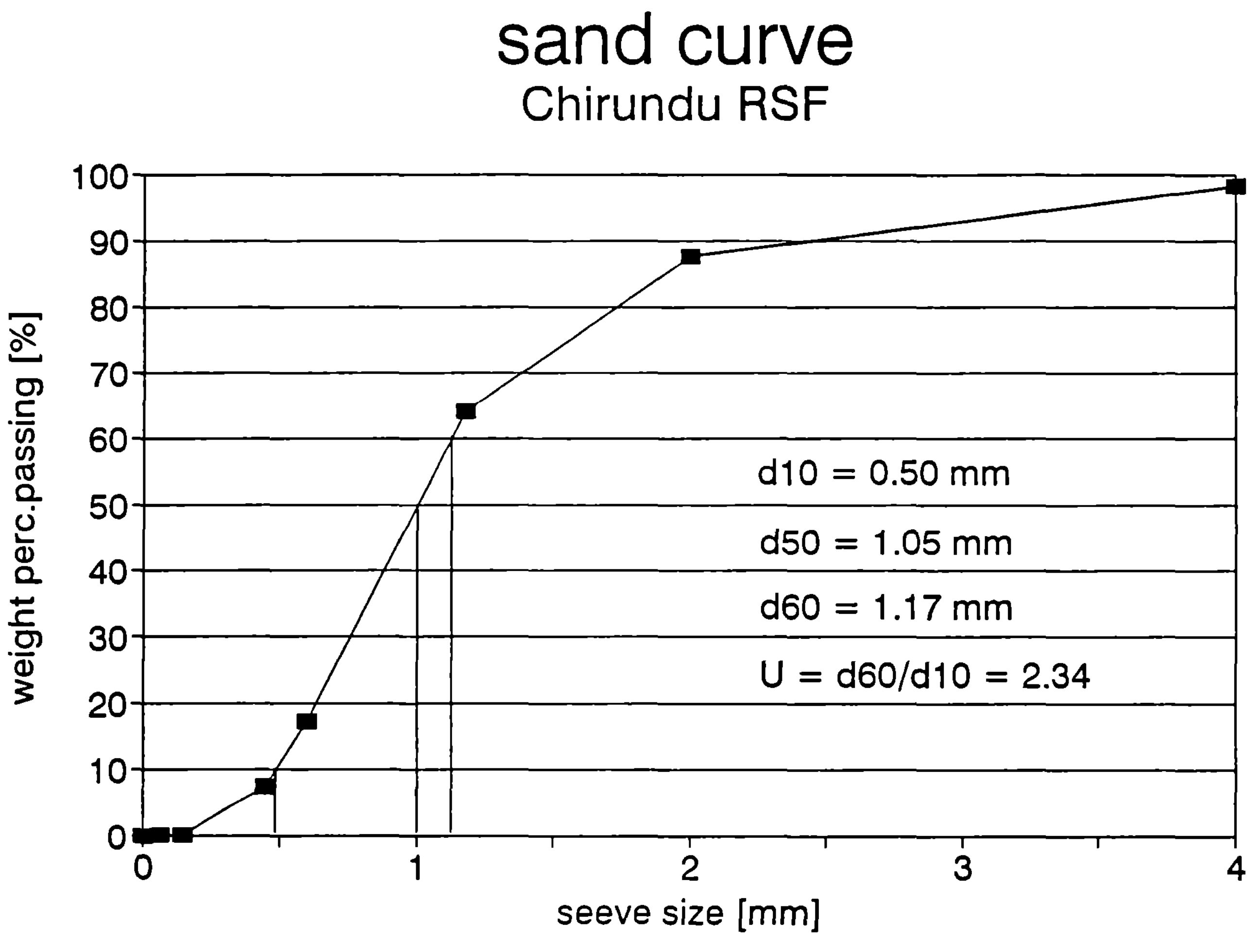
APPENDIX D

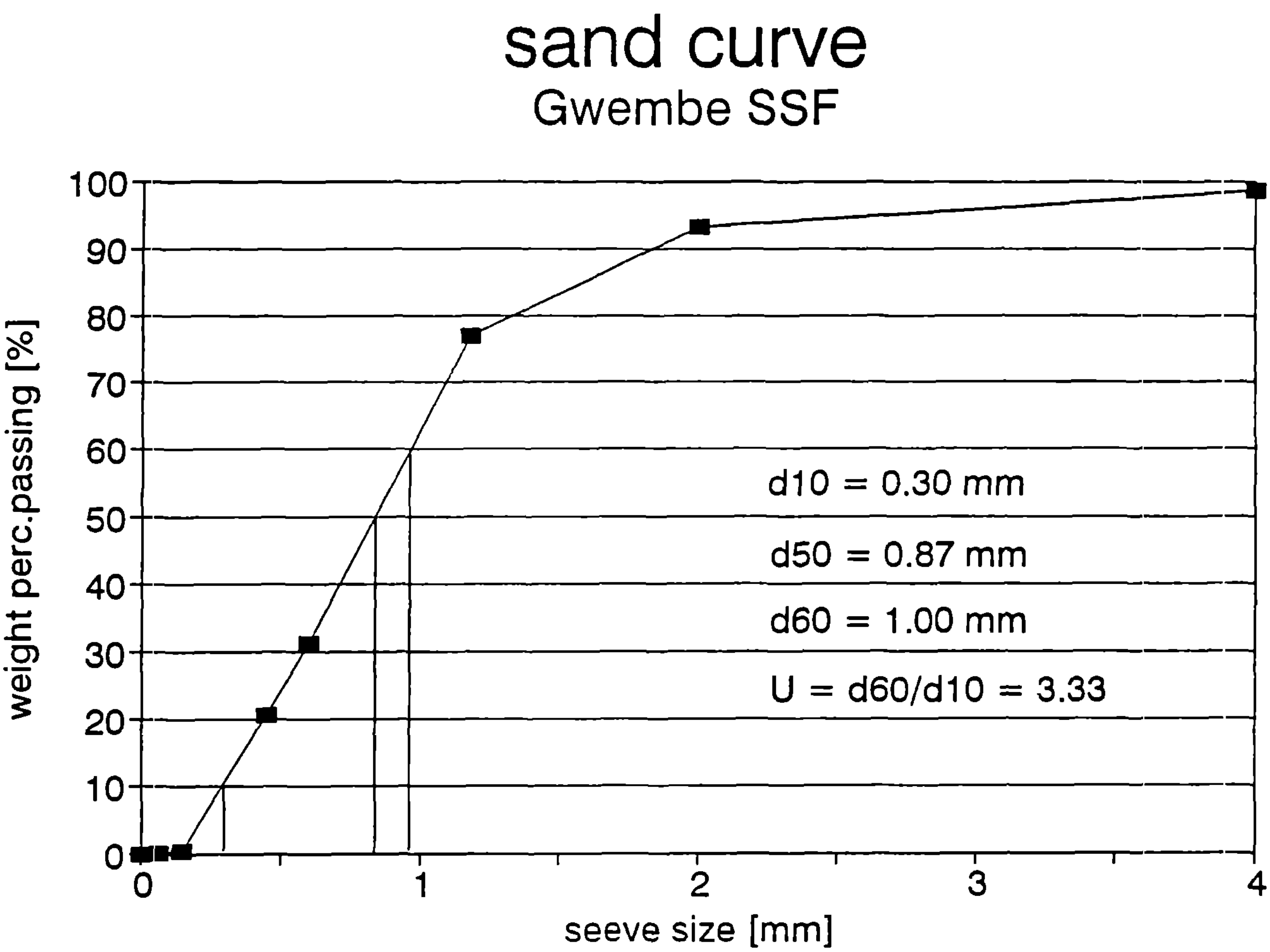
SANDCURVES

1 SANDCURVES

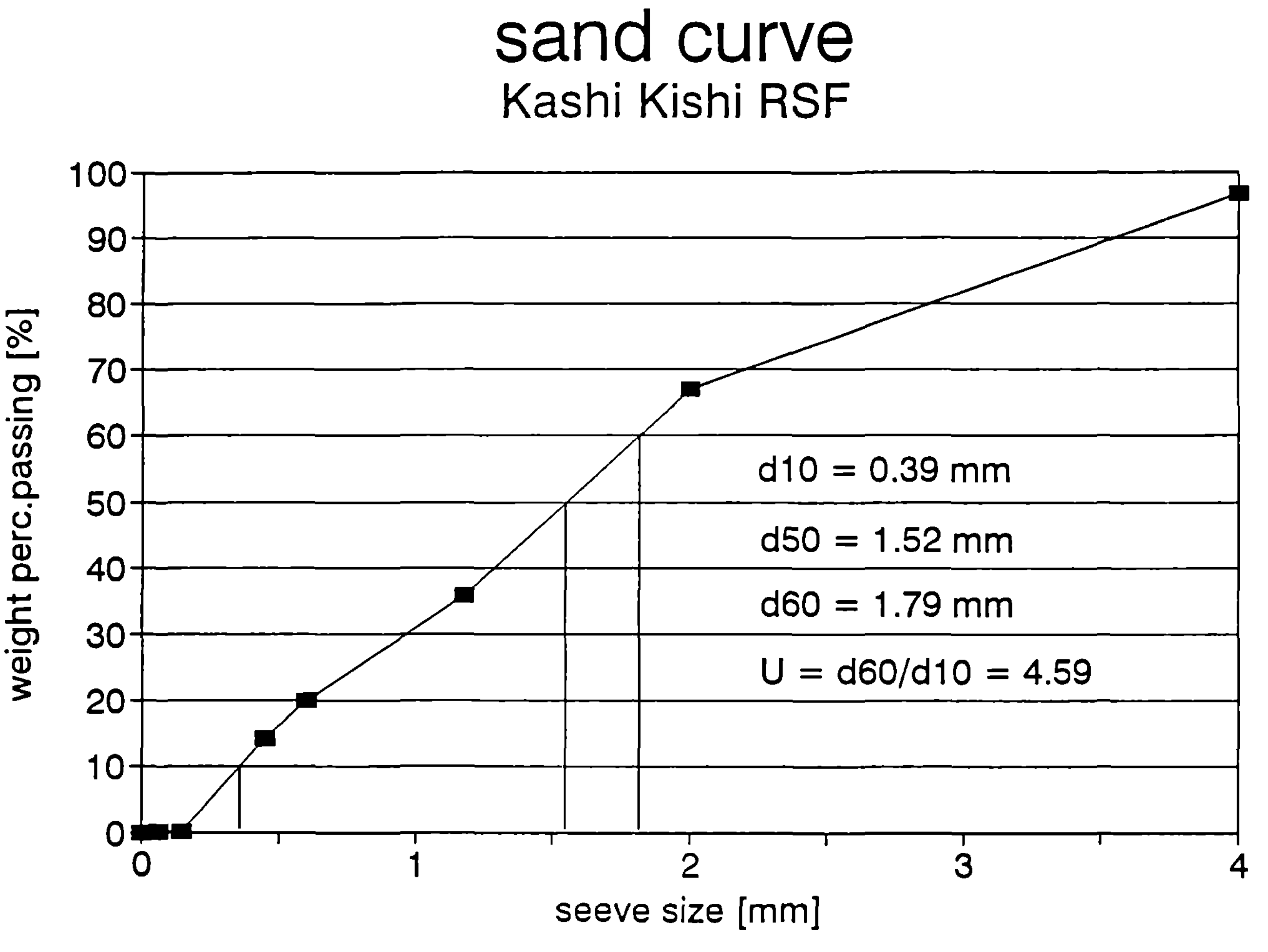

sand curve
Chadiza SSF
100
90
80
70
60
50
40
30
20
10
0
weight perc.passing [%]
d10 = 0.40 mm
d50 = 1.25 mm
d60 = 1.63 mm
U = d60/d10 = 4.10
0
1
2
3
4
seeve size [mm]

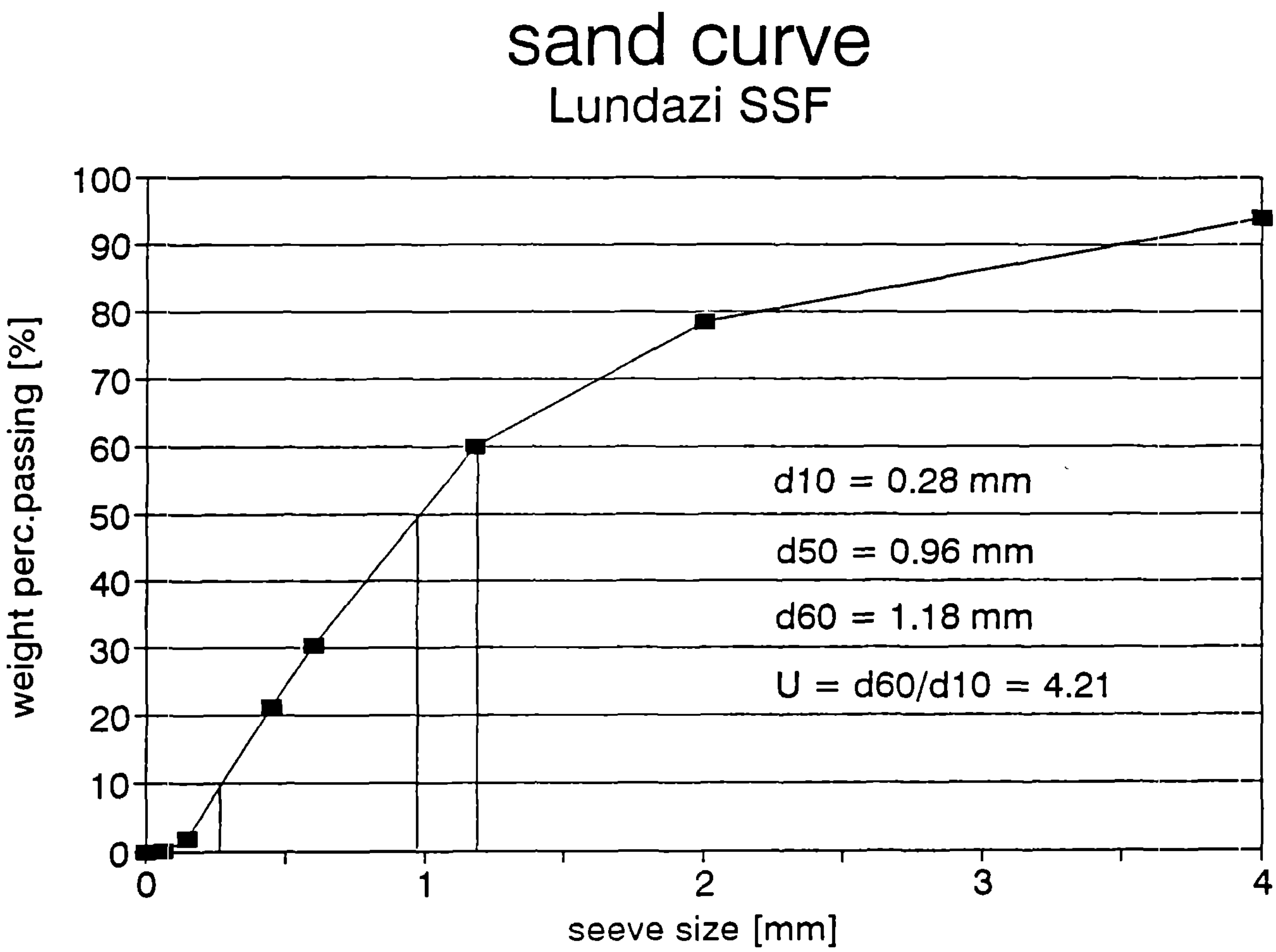

sand curve
Chipata RSF, new mixed sand
100
90
80
70
60
50
40
30
20
10
0
weight perc.passing [%]
d10 = 0.85 mm
d50 = 1.40 mm
d60 = 1.52 mm
U = d60/d10 = 1.79
0
1
2
3
4
seeve size [mm]

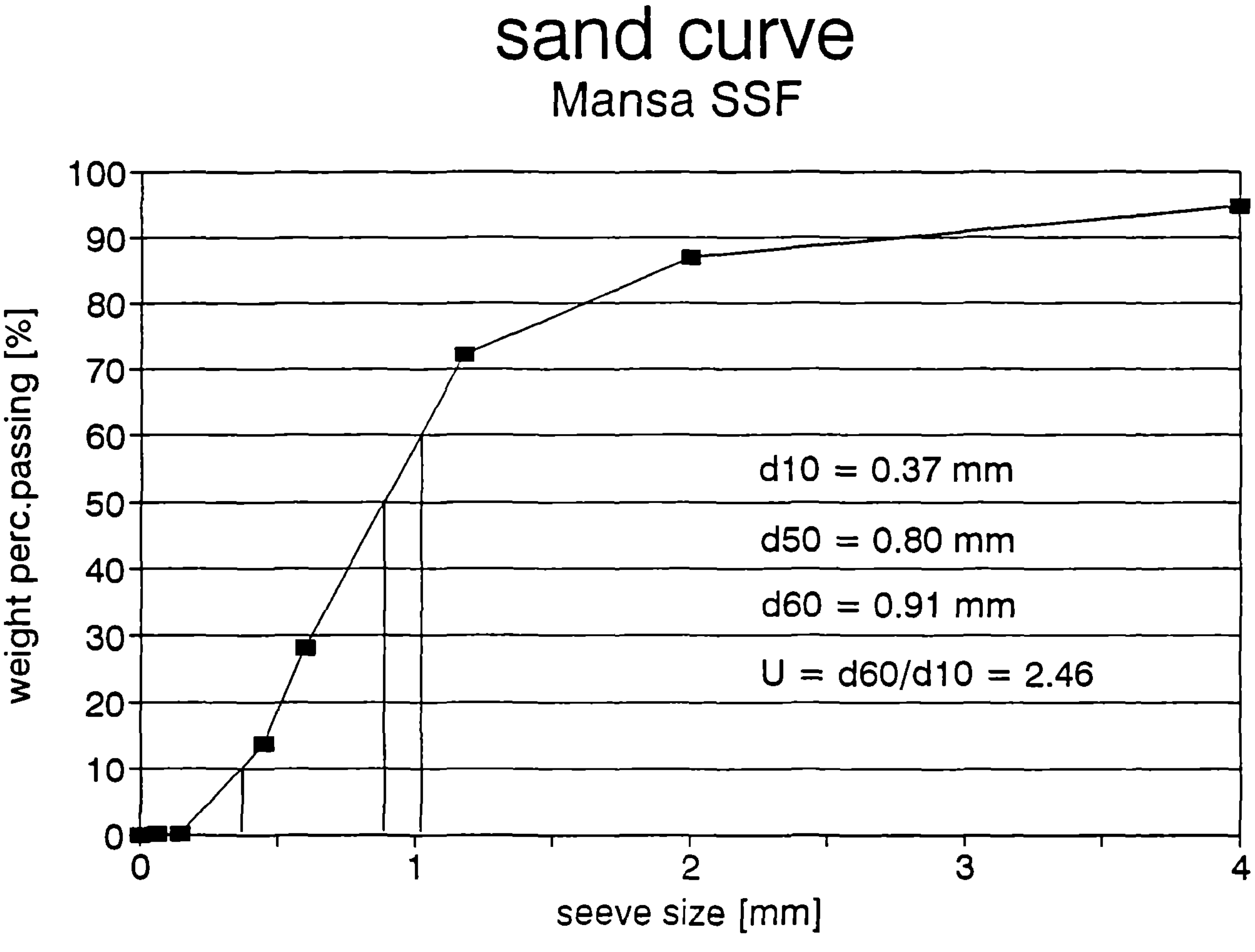

sand curve
Chirundu RSF
100
90
80
70
60
50
40
30
20
10
0
weight perc.passing [%]
d10 = 0.50 mm
d50 = 1.05 mm
d60 = 1.17 mm
U = d60/d10 = 2.34
0
1
2
3
4
seeve size [mm]

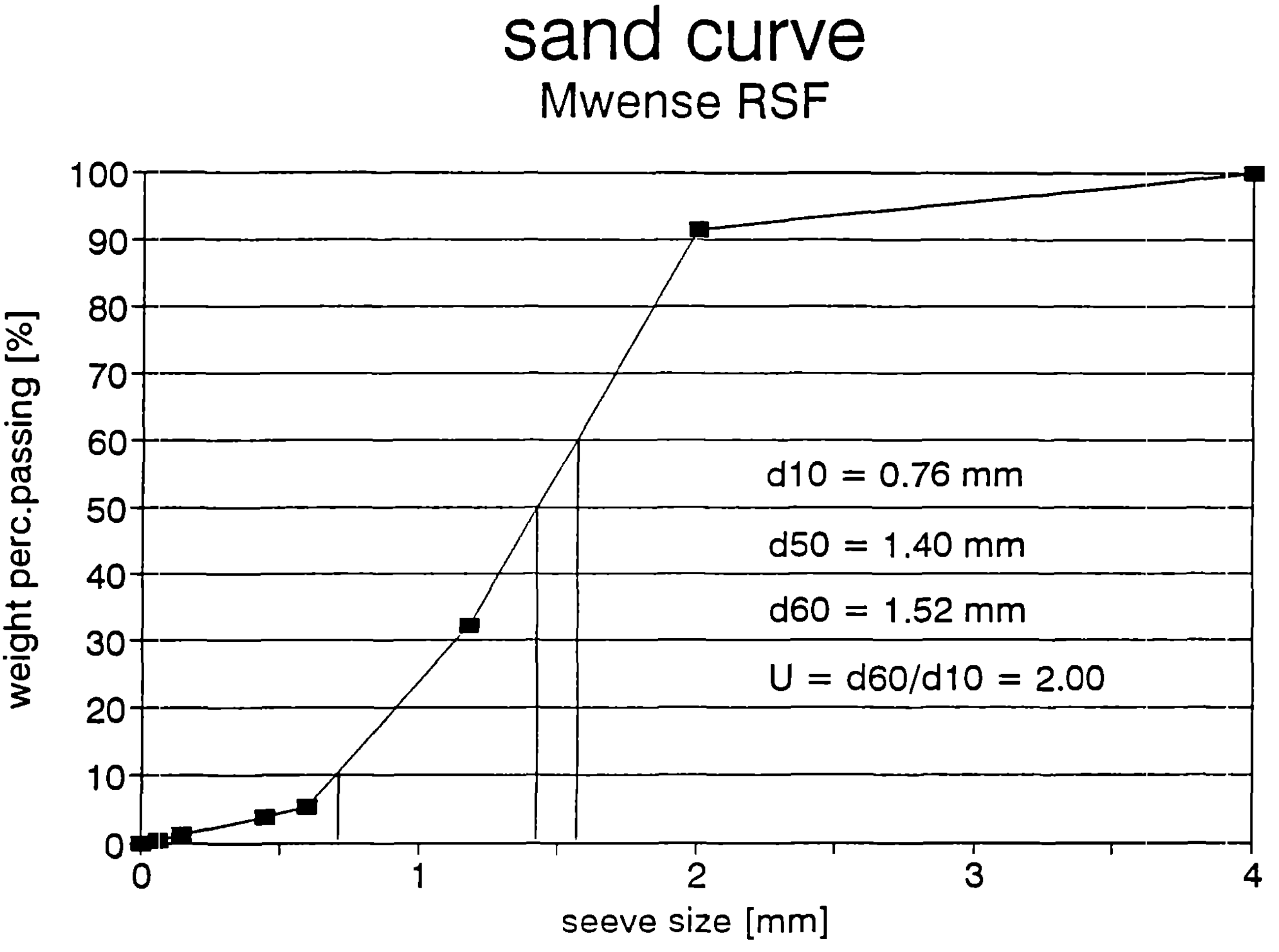


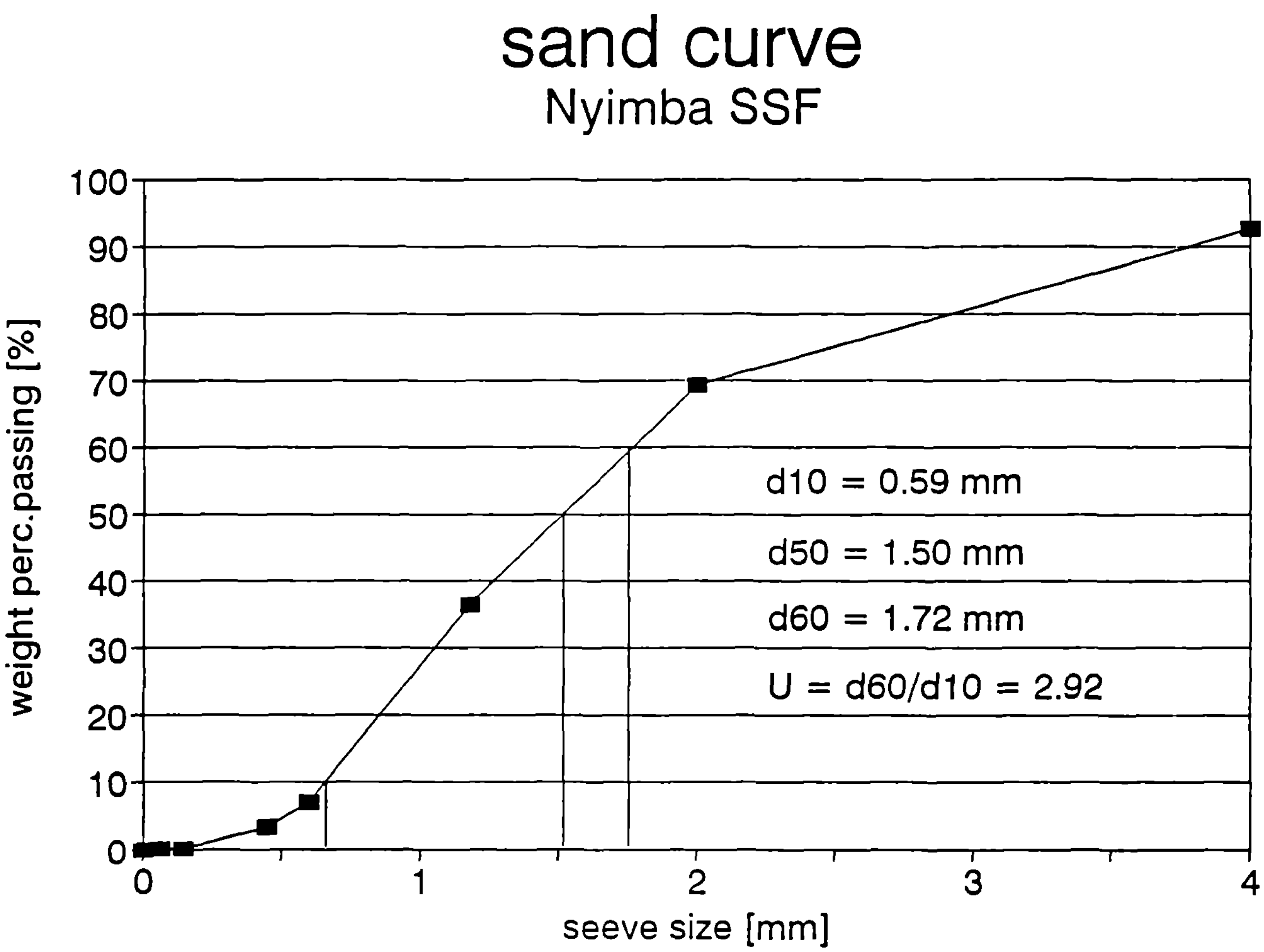
sand curve
Gwembe SSF
weight perc.passing [%]
100
90
80
70
60
50
40
30
20
10
0
d10 = 0.30 mm
d50 = 0.87 mm
d60 = 1.00 mm
U = d60/d10 = 3.33
0
1
2
3
4
seeve size [mm]

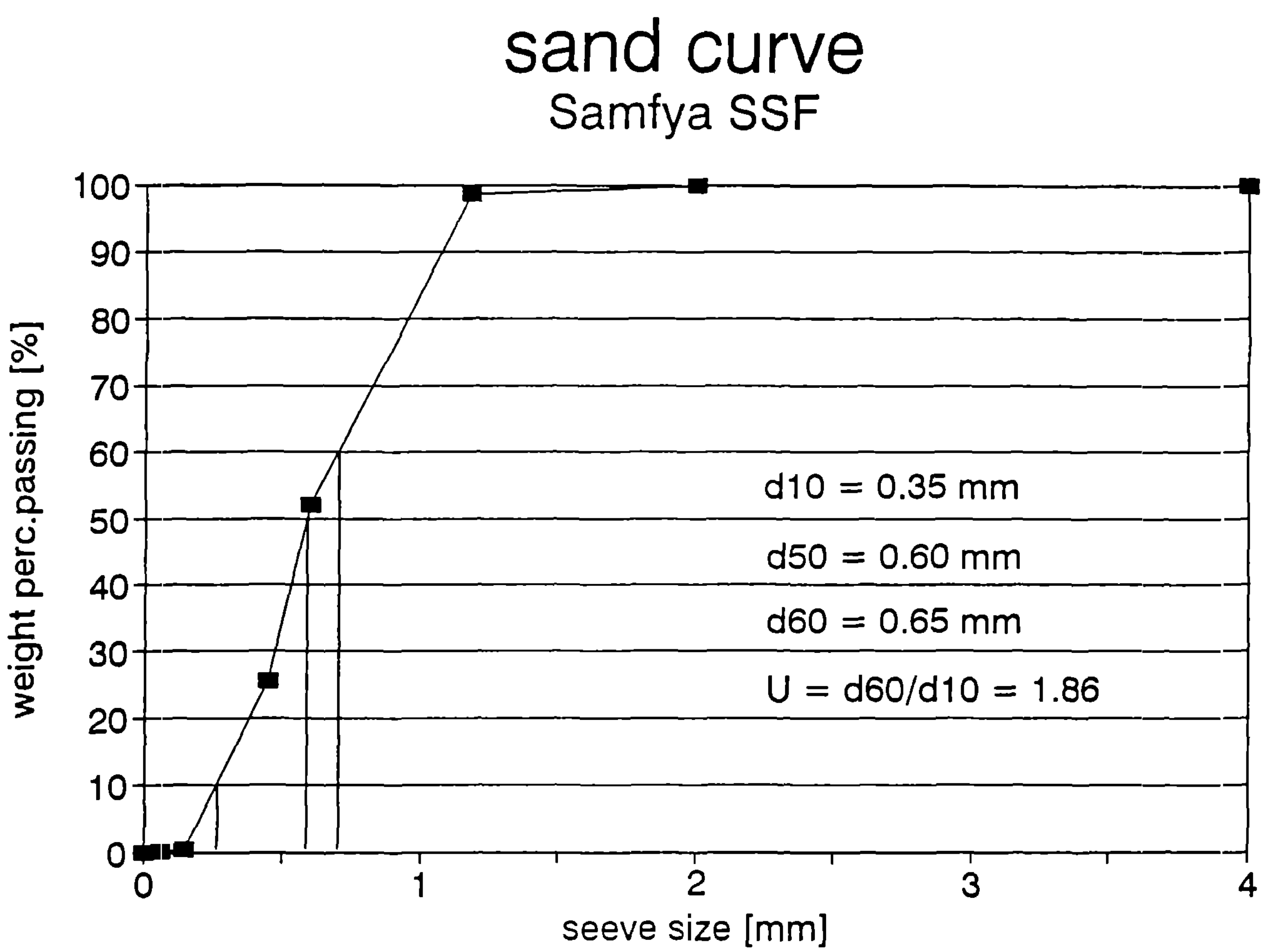

sand curve
Kashi Kishi RSF
weight perc.passing [%]
100
90
80
70
60
50
40
30
20
10
0
0
1
2
3
4
seeve size [mm]
d10 = 0.39 mm
d50 = 1.52 mm
d60 = 1.79 mm
U = d60/d10 = 4.59

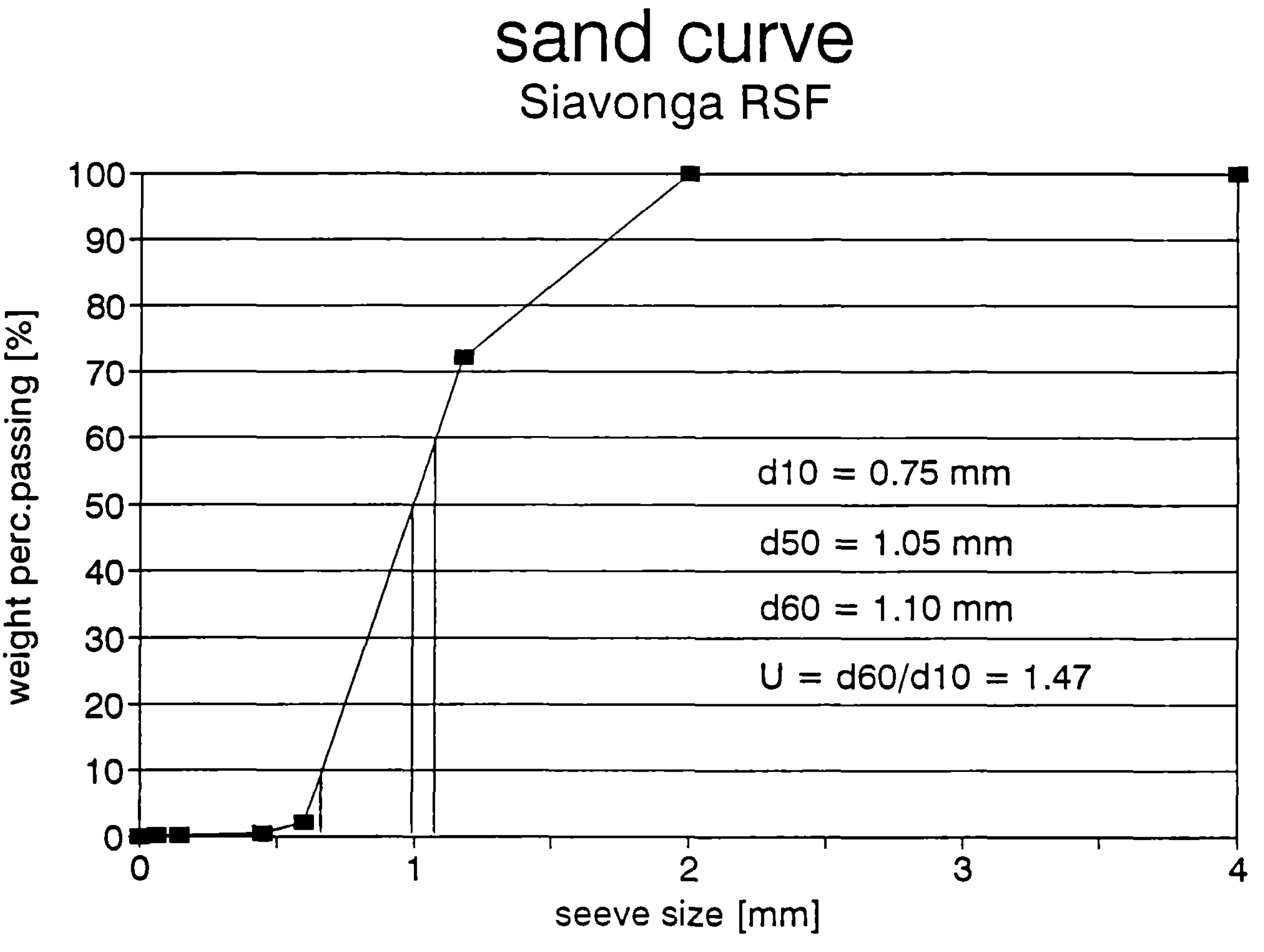

sand curve
Lundazi SSF
100
90
80
70
60
50
40
30
20
10
0
weight perc.passing [%]
0
1
2
3
4
seeve size [mm]
d10 = 0.28 mm
d50 = 0.96 mm
d60 = 1.18 mm
U = d60/d10 = 4.21

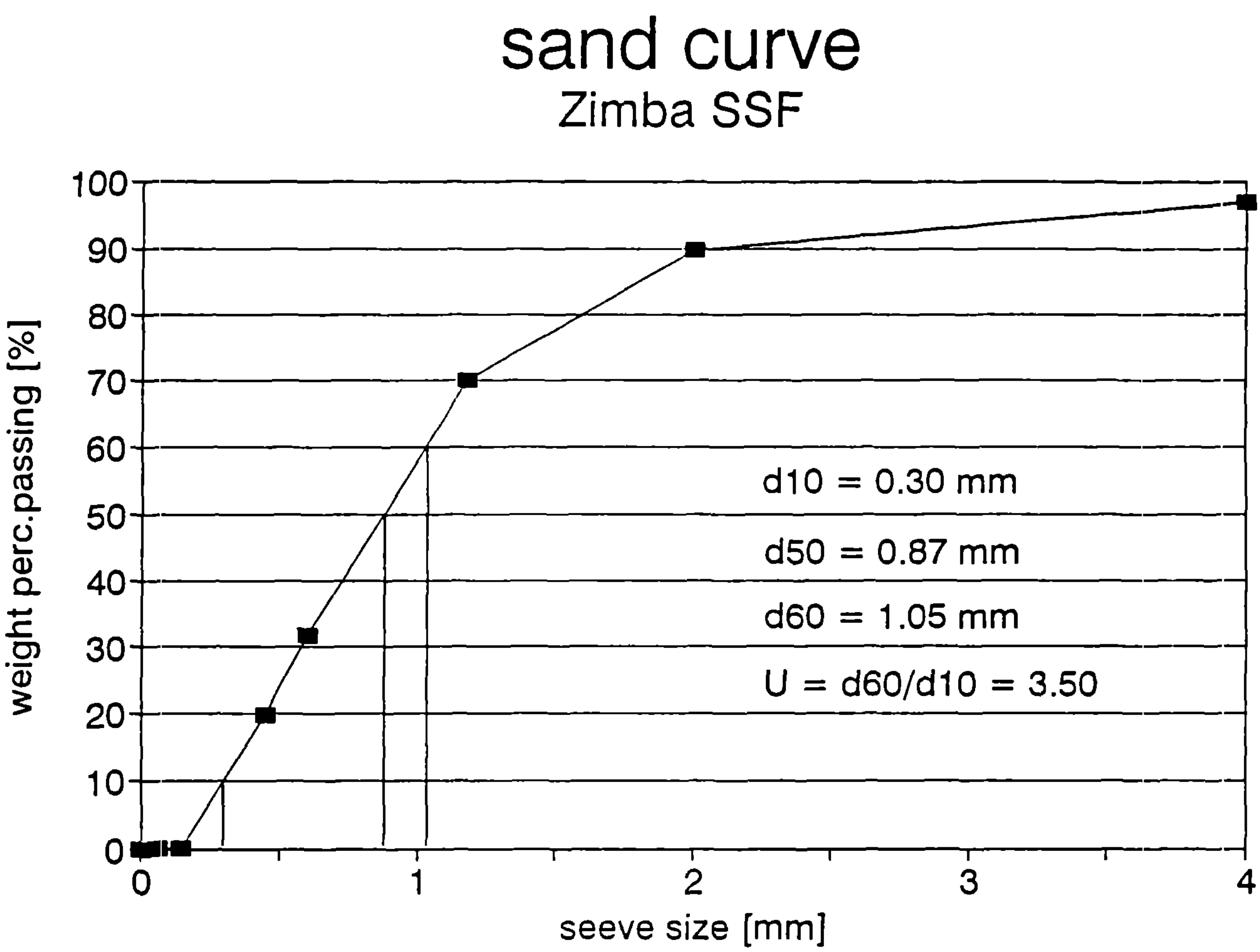

sand curve
Mansa SSF
100
90
80
70
60
50
40
30
20
10
0
weight perc.passing [%]
d10 = 0.37 mm
d50 = 0.80 mm
d60 = 0.91 mm
U = d60/d10 = 2.46
0
1
2
3
4
seeve size [mm]


sand curve
Mwense RSF
weight perc.passing [%]
100
90
80
70
60
50
40
30
20
10
0
0
1
2
3
4
seeve size [mm]
d10 = 0.76 mm
d50 = 1.40 mm
d60 = 1.52 mm
U = d60/d10 = 2.00


sand curve
Nyimba SSF
weight perc.passing [%]
100
90
80
70
60
50
40
30
20
10
0
0
1
2
3
4
seeve size [mm]
d10 = 0.59 mm
d50 = 1.50 mm
d60 = 1.72 mm
U = d60/d10 = 2.92


sand curve
Samfya SSF
weight perc.passing [%]
100
90
80
70
60
50
40
30
20
10
0
0
1
2
3
4
seeve size [mm]
d10 = 0.35 mm
d50 = 0.60 mm
d60 = 0.65 mm
U = d60/d10 = 1.86


sand curve
Siavonga RSF
weight perc.passing [%]
100
90
80
70
60
50
40
30
20
10
0
0
1
2
3
4
seeve size [mm]
d10 = 0.75 mm
d50 = 1.05 mm
d60 = 1.10 mm
U = d60/d10 = 1.47


sand curve
Zimba SSF
weight perc.passing [%]
100
90
80
70
60
50
40
30
20
10
0
0
1
2
3
4
seeve size [mm]
d10 = 0.30 mm
d50 = 0.87 mm
d60 = 1.05 mm
U = d60/d10 = 3.50

APPENDIX E

OPERATION SHEETS DWA

DEPARTMENT OF WATER AFFAIRS — TOWNSHIP WATER SUPPLIES

I

Province: ______________________ Township: ______________________

Monthly billing (Kwacha) Month / Year: ______________________

		Billed revenues (Kwacha)			Collected revenue (Kwacha)	Accumulated uncollected revenue (Kwacha)	Billed water (m^3)	
		Nos	Rate	Total			Per connection	Total
A Unmetered connections (flat rates)	1 High cost houses							
	2 Medium cost houses							
	3 Low cost houses							
	4 Single tap houses							
	5 Communal taps							
	6 GRZ - institutions							
	7 Parastatals, industrial and commercial							
B Metered connections	1 Houses							
	2 GRZ - institutions							
	3 Parastatals, industrial and commercial							
C New connections	4 Charged at estimated costs							
D Disconnections	All groups							
E Reconnections	1 Private consumers							
	2 GRZ - institutions							
	3 Parastatals, industrial and commercial							
TOTALS								

Notes:

Amounts rounded to nearest Kwacha

Remarks:

Date: ______________________

Sign: ______________________

Name: ______________________

DEPARTMENT OF WATER AFFAIRS			TOWNSHIP WATER SUPPLIES	
2	Province: ______		Month / year: ______	
	INCOME / EXPENDITURE		Township: ______	

Type	This month		Accumulated this year	
	Budget	Actual	Budget	Actual
Billed water in m^3 (A)				
Water production in m^3 (B)				
Billed revenues ZK (C)				
Collected revenues (D)				
Uncollected revenue (E)				
Billing efficiency (F) $F = \frac{A}{B} \cdot 100$ (%)				
Collection efficiency (G) $G = \frac{D}{C} \cdot 100$ (%)				
Revenue efficiency $H = F \times G$ (%)				
Operational costs 1. Labour costs including salaries and allowances				
2. Power costs				
3. Chemicals				
4. Spares / consumables				
5. Transport costs				
6. Miscellaneous				
Total operational costs (I)				
Cost recovery $\frac{D}{I} \cdot 100$ (%)				

3

Province: ____________

Year: ____________

SUMMARY OF INCOME / EXPENDITURE

Month: ____________

INCOME	Water Supply Scheme						
Water production m^3							
Billed water m^3							
Billed revenue ZK							
Collected revenue ZK							
COST							
Salary including allowances							
Other direct running costs (2 — 6, form 2.)							
Total operational cost							
Arrears							

Remarks:

Date:	Sign:	Name:

APPENDIX F

ZAMBIA, THE COUNTRY

1 GENERAL

Zambia is a landlocked country in Southern Africa with over eight million inhabitants. Zambia, whose name derives from the Zambezi river, was formerly known as Northern Rhodesia. It covers an area of 752,614 km^2. The country is bordered by Zaire, Angola, Namibia, Botswana, Zimbabwe, Mozambique, Malawi and Tanzania. Zambia consists mainly of high plateau which is deeply entrenched by the Zambezi river and its tributaries, the Kafue and the Luangwa. There are three distinct weather season: a cool dry season from May to August, a hot dry season from September to November and a hot rainy season from December to April. Zambia is a typical savannah country. Outside the urban areas most of the country is sparsely populated because many people move to the towns in search of employment.

Lusaka, the capital, has a population of more than 700,000 and covers an area of 360 km$_2$. It came into existence in 1905 as a site near the railway and became capital of the country in 1931. Due to extensive growth of the population, there is a major shortage in work and housing facilities although both light and heavy industry has expanded in Lusaka. Various industries are present such as electrical and mechanical engineering, metal fabrication, breweries and industries producing textiles, chemicals, glass and plastics.

2 POLITICAL SITUATION

The republic of Zambia has gained Independence in 1964. The political ideology of the government was based upon Zambian Humanism emphasizing human and racial equality, and participation by the people. The republic has been ruled by president Kaunda form 1964 until 1991.

Since 1972 Zambia had been a one-party participatory democracy state, when the constitution of the republic was amended to provide that there should only be one legal political party, the United National Independence Party (UNIP). In October 1991 there have been multi-party elections for the first time in the history of Zambia. The UNIP got just 23 of the 150 parliament chairs. Winner was Frederick Chiluba of the Movement for Multi-party Democracy (MMD) who is now president of Zambia.

3 ECONOMIC AND SOCIAL SITUATION

Zambia has a low income economy: a GNP per Capita of US $ 250 (1987) with a negative average growth rate of 5,6% and a public debt/GNP - ratio which is one of the highest in the world. Prices of copper declined steeply during the past decades and as a result the amount of money available for investment and economic development is reduced. The inflation of the national currency, the Kwacha, is high. About 35% of the people of Zambia derive a living directly from agriculture. The agriculture society stands second only to copper mining in importance to the national economy. Because of the fact that the prices of mealiemeal, which is the staple food in Zambia, have artificially been kept low by the government for several years, farmers did not get enough money for their maize crop and therefore tried to grow other, more rewarding crops. This resulted in a shortage of mealiemeal. This year, 1992, negotiations resulted in reasonable mealiemeal prices for farmers for next year.
Last rainy season was the driest in about twenty years. Only approx. 20 percent of the crops could be gathered.

Compared to the costs of living salaries remain very low, which results in a high pressure on the new Government to stop the inflation and rising food prices.