ÅBO AKADEMI UNIVERSITY
FACULTY OF SCIENCE AND ENGINEERING

Proposal of a long-term water supply network to Honde Valley Ward 10

Master's Thesis
by
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Carried out at the Laboratory of Process and Systems Engineering at Åbo Akademi University under the supervision of docent Frank Pettersson.
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Abstract

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The thesis provides a technical framework of gravitationally fed water distribution systems in rural developing regions. The functionality of gravity-fed networks' theory includes energy profiles in the network, pressures and head losses, and demand and supply patterns which are needed for the dimensioning of the components. The components included in network systems are reviewed and how to estimate their sizes is presented. Furthermore, basic water quality analyses with respective treatment methods for centralized systems are also reviewed.

The theory is applied to a project proposal of a gravity-fed distribution system in Honde Valley Ward 10 in Zimbabwe. A group of students from InnoEnergy (KTH, UPC) conducted a field trip in 2017 to the area where multiple topics with potential for development were analyzed, whereas the priority was given to the improvement of the sanitation and water supply situation in Ward 10. The public pipeline built in 2003 is deteriorating and only few inhabitants have access to it. Consequently, unorganized self-made haphazardous systems have been built for individual use. The goal of the thesis is to provide a technical basis for how to develop an improved public system in the area in order to controllably manage the water resources and provide a reliable water supply to the inhabitants.

The area has been mapped remotely with an open-source software, QGIS, based on satellite pictures and interviews. The water sources have been surveyed with a topographical runoff simulation in QGIS and verified with local measurements, whereafter a new proposed distribution system has been drawn and dimensioned according to the presented theory using data provided by Zimbabwe National Water Authority (ZINWA) and Fortune Development Center (FDC). To verify the functionality of the network from a hydrological perspective the proposed system has been simulated on MIKE+ modelling software by DHI.

The result from the simulations shows that Ward 10 is technically suitable for a gravity-fed water distribution network, however, some approximations have been used that need further on-site data and verifications to be more accurate. Although the area is technically suitable, there are other aspects that need to be considered. For example analysis of the state of the current pipelines and an improved maintenance and management system for the water supply infrastructure need to be developed.

Keywords: Water supply, gravity-fed distribution, water quality, potable water, water treatment, developing region

engineers without borders SWEDEN

Preface

This Master's thesis is part of an Engineers Without Borders project to develop the water supply standards in Honde Valley Ward 10.

I want to thank my office mates and friends who have brought an everyday joy to the lunch and coffee breaks in-between the workhours spent on this project. Special thanks to my thesis supervisors Frank, Linnéa and Johanna who have enthusiastically shown interest in the work and supported me throughout the project, and additionally a thank you to Jacob for providing valuable information to the project and showing great appreciation, which has helped me stay motivated. Final thanks to my brother, M.sc Geology and Mineralogy Mathias Eriksson, for doing the runoff simulation in QGIS and helped me get started with the mapping software.

I hope the villagers of Ward 10 and successors to this project will find some use of the provided theory and parts of the proposal to further the development in Ward 10 Honde Valley.

"He blir nou bra"

Jeremias Eriksson
Turku, 12^{th} January 2022

Abbreviations

BPT	Brake-Pressure Tank
CDBP	Chlorination Disinfectant By-Product
CT	Contact Time
EC	Electro Conductivity
EWB	Engineers Without Borders
FDC	Fortune Development Center
HRT	Hydraulic Retention Time
HT	Header Tank
HW	Hazen-Williams
LIDAR	Light Detection and Ranging
mWG	meter Water Gauges
NTU	Nephelometric Turbidity Units
PN	Nominal Pressure
SODIS	Solar Water Disinfection
SSF	Slow Sand Filter
ST	Storage Tank
TDS	Total Dissolved Solids
TSS	Total Suspended Solids
TVC	Total Viable Count
ZINWA	Zimbabwe National Water Authority

Simulation and system legends

Code: **AXabcYy**

AX Line ID

OP – Old Pipeline (existing pipeline)
L1 – Line 1 (existing pipeline)
L2 – Line 2 (existing pipeline)
L3 – Line 3 (existing pipeline)
L4 – Line 4 (existing pipeline)
L5 – Line 5
L6 – Line 6
L7 – Line 7
L8 – Line 8
L9 – Line 9

abc Village where the component is located

MaW – Matingo West of Mupenga River
MaE – Matingo East of Mupenga River
Chi – Chipunha
Nya – Nyakabinga
SG – Saruwaka Gowa
Mut – Mutsaka
Sam – Samaringa
Bar – Baradza
Ham – Hambira
Ngw – Ngwende
Mte – Mutetwa
Man – Manyonho
Nyb – Nyabadza
Mts – Mutsamba

Yy Number of node or pipe from starting point (1-13), or if the node is a reservoir it has the following ID:

HT – Header tank
PBT – Pressure breaker tank
ST – Storage tank
Intake – intake to the system

Additional legends

Code: **AX(A/B)abcY(.Zo/Dummy)**

A / B Secondary main line (A or B)

.Z Branch from main line (1-5)

o Outlet node with set demand

Dummy Theoretical demand node, which input equals the nearby tank's output

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1 Introduction

This project is part of a more extensive program to develop the living standards in Honde Valley Ward 10, Zimbabwe. In developing countries, sustainability may be suppressed by more urgent pressures of the control for the situation regarding economic survival or population growth, as well as a lack of understanding and education. *Our common future* by the Brundtland reports defined sustainable development as:
"development that meets the needs of the present without compromising the ability of future generations to meet their own needs." However, solutions to environmental problems are not without cost and, thus, a choice between multiple options must be made with an understanding of basic economic principles. Hence, proposals such as this one must be reviewed with other possible options and their feasibility determined from the local end users' perspective.
In 2017, a survey was conducted by students from InnoEnergy (KTH and UPC) where multiple aspects of the living standards in the village were analyzed, where the focus was on the following topics:

- water supply harmonization, purification, and sanitation
- sustainable and clean energy access
- roads and infrastructure improvement
- agriculture
- community services
- ecotourism

Water supply was prioritized as the first topic to improve. The sanitation standards are primitive and access to purified water is lacking. Currently, the purification of the water is done on an individual level in Ward 10 (Chikuhwa, 2021). Poor sanitary conditions with a combination of poor nutrition can have disastrous effects on the community when water-related diseases rapidly spread throughout the village (Tebbutt, 2002). Ward 10 is developing a "green belt" program where the water supply for irrigation is focused on. This will help the inhabitants provide a livelihood from exporting products from their agriculture. Therefore, water distribution in large quantities with relatively good quality is essential to improve. A reliable method is through a pipelined network with a centralized treatment.

A theoretical framework is presented in the thesis of how gravitationally distributed water systems work, i.e. how the pressure profiles and demand patterns determine the components' dimensions of the system. The components are tanks, pipes and filters which are reviewed, and rough dimensioning of their sizes is presented. The presented theory is applied to the situation in Honde Valley Ward 10, with a goal to improve the water supply situation for the inhabitants. The situation of the area is reviewed based on the data provided by FDC, ZINWA and the previous report. Mapping of the village borders and water resources has been made with an open-source software, QGIS. The maps are imported to a hydraulic simulation software MIKE+ WD, to prove the proposed network's hydraulic functionality. Technical details and management plans need to be verified from on-site surveys at a later stage of the project.

2 Theory of gravitationally assisted water distribution systems

A distribution network should be built where there is a concentrated population; otherwise, excessive pipe length must be laid out, and other means of collecting water might be more efficient. Two conditions must be met for a gravity distribution network to work. Firstly, the water quantity from the sources must be equal to or larger than the water demand. Secondly, at least a 1% slope from the water source or header tank to the end node of the system is needed for a gravity-fed distribution, i.e., the topographic energy profile. If these two conditions are met, then theoretically, a gravity-driven system can be built (Sampers, 2005).

Some general aspects for distribution networks in developing countries need to be considered when designing the system. It needs to be relatively simple to construct. The elements should be able to be repaired with local technologies. If a system is built with imported parts, spare parts can be difficult to obtain and the whole network might be left malfunctioning or unused. Therefore, the components should be made of materials which can be obtained from the local markets. Using local resources also reduces the cost of construction, which is another crucial criterion (WHO, 2017).

The whole system should be easily operated, maintained, and supervised without needing professional training or education. If a problem in the process line occurs, it should be able to be resolved by the locals to keep the process continuously running. In some areas where electricity is expensive, it could also be necessary to have a process that does not need electricity continuously. Sometimes the area lacks infrastructure or is otherwise inaccessible; therefore, minimal machinery should be required to construct the facilities, and no electricity to operate the system can be an essential aspect (WHO, 2017).

2.1 Energy profiles

The topographic profile describes the energy balances in the system. It is a good custom to draw the profile for each line and include the line's elements in the profile to obtain a summary of the line's functionality. For the topographic survey, a LIDAR map can be used as a rough estimation. For a more detailed study of the elevation for the system, an on-site survey should be done with the use of a theodolite, or alternatively, an Abney level or clinometer in inaccessible areas (Sampers, 2005).

Bernoulli's equation (1) describes the energy principle in gravity-fed distribution systems: the energy at the beginning of the system equals the sum of the energy at the end of the system and the friction losses. It can be divided into three parts, the velocity in the system ($\frac{1}{2} \cdot \rho \cdot V^2$), the elevation of the system ($\rho \cdot g \cdot h$) and the pressure in the system (p), which should all equal a constant energy level (Q_c).

$$\frac{1}{2} \cdot \rho \cdot V^2 + \rho \cdot g \cdot h + p = Q_c \tag{1}$$

The pressure in water-distribution systems is usually expressed in water columns as meter water gauges (mWG), i.e., the weight of the water above the considered point.

$$\frac{1\text{kg}}{\text{cm}^2} = 1 \text{ bar} = 10 \text{ mWG} \tag{2}$$

2.1.1 Static pressure

Static pressure is the pressure profile of the water energy when all the outlets are closed, and the flow is zero. In simple systems, this should be the maximum pressure in the pipes, which determines the pressure limits and material choices for the pipes. The pressure load in closed pipes is regarded as the water column in meters between its upper point and the point of consideration. The static pressure (p_{st}) can be described as equation 3, where (H) is the elevation difference from the two measured points (Sampers, 2005).

$$p_{st}(mWG) = H(m) \tag{3}$$

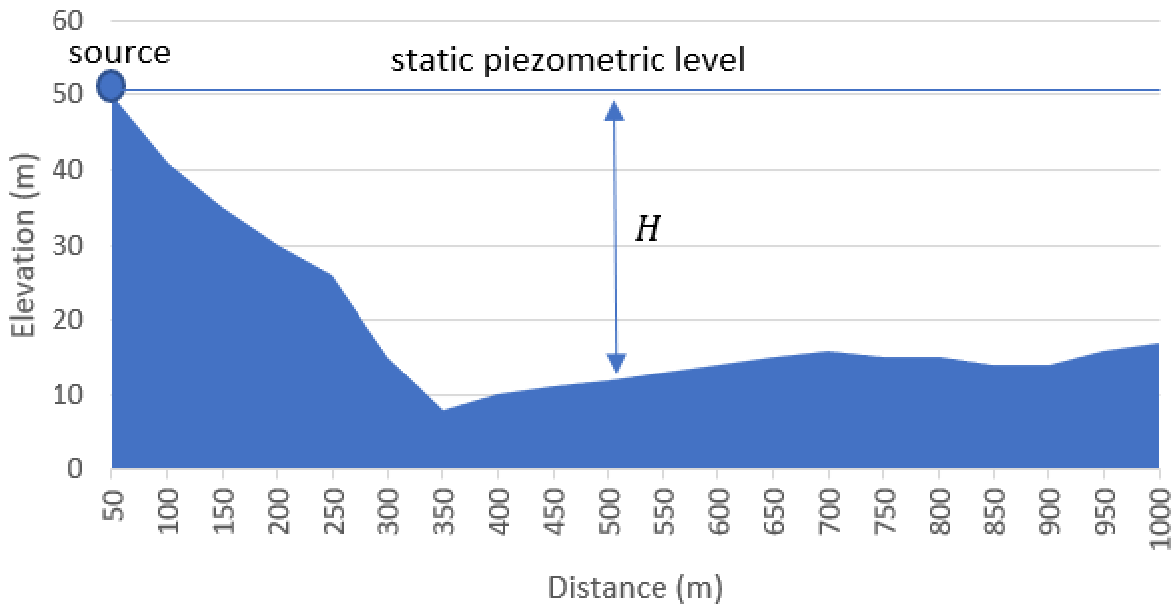

Figure 1, static pressure gradient

Nominal pressure (*PN*) is the pressure the chosen pipes should withstand without rupture, which in a static-head profile is the same as the static pressure. Standardized sizes range from PN6 up to PN400.

2.1.2 **Dynamic pressure**

The dynamic pressure is the pressure occurring when valves and taps are open, and a flow in the pipes is present due to head losses in the pipes when the water is moving (see chapter 2.1.3). The dynamic pressure (p_{dyn}) is therefore described as the static piezometric level (H) subtracted by the head losses (Δp), as depicted in Figure 2 and equation 4.

$$p_{dyn} = H - \Delta p \tag{4}$$

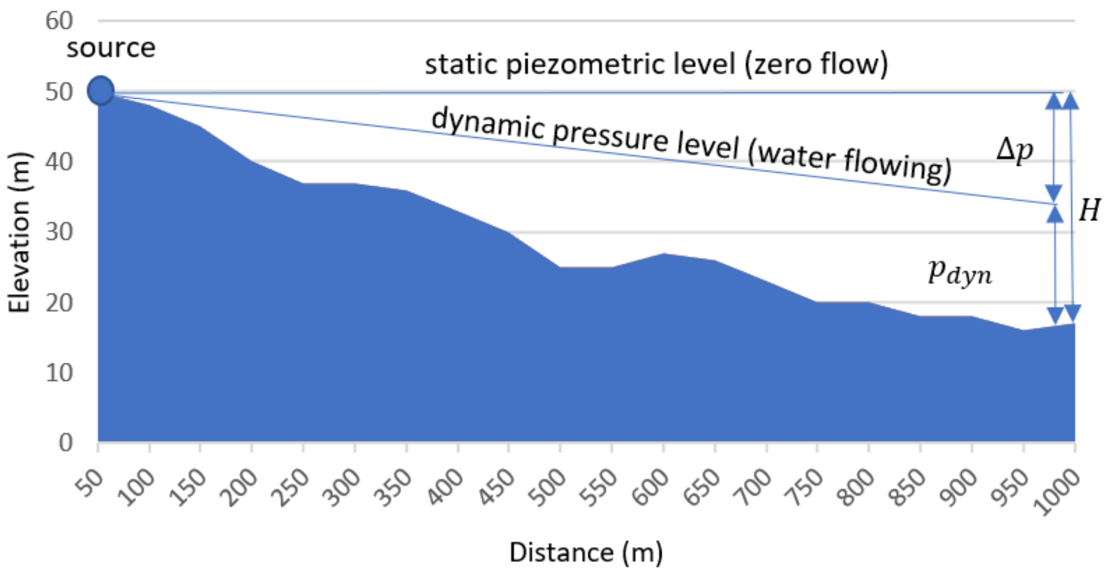

Figure 2, dynamic pressure gradients

2.1.3 Head losses

The linear head losses in a pipe depend on several things: the pipe diameter, the flow in the pipe, the length of the pipe, and the roughness of the pipe material. As a rule of thumb, a head loss coefficient of 1% corresponds to a loss of 1 mWG for a 100-meter-long pipe. For a more precise calculation, head loss formulas have been developed. The most used friction loss formula used in the US is the Hazen-Williams (see equation 5). It can only be used for water, not other liquids, and was initially developed for turbulent flows only (Sampers, 2005).

$$H_L = \frac{4.727 \cdot L \cdot Q^{1.852}}{C^{1.852} \cdot d^{4.871}} \tag{5}$$

where L is the pipe length, Q is the flow, d is the inner diameter of the pipe, and C is the resistance coefficient which varies depending on which pipe material is used, according to Table 1.

Table 1, Hazen-Williams constants

Pipe material	HW coefficient, unitless (C)
Cast iron	130-140
Concrete or concrete-lined	120-140
Galvanized iron	120
Plastic	140-150
Steel	140-150
Vitrified clay	110

2.1.4 Secondary head losses

Secondary head losses are the friction from when water runs through fittings, valves, junctions, etc. Secondary head losses are generally only about 10% of the linear head losses and are usually neglected in simple networks (Sampers, 2005).

2.2 Base demand

The estimated lifetime of the network system should be around 10-30 years; therefore, population growth needs to be considered when designing the demand flow. Usually, a 30-year anticipation of population growth gives a relatively good ratio between the cost and the result. Equation 6 can be used to estimate the population growth.

$$P_f = P_0 \cdot \left(1 + \frac{r}{100}\right)^t \tag{6}$$

where P_f is the future estimated population, P_0 is the current population, r is the local growth ratio, and t is the time perspective. (Arnalich, 2020)

The amount of water per capita needed is illustrated in figure 3. For a long-term solution of a pipelined water system, the demand per inhabitant should be at least 60-80 liters per capita per day of water in developing regions, whereas $5 - 10\,\mathrm{l/m^2}$ is used for crop (WHO, 2017).

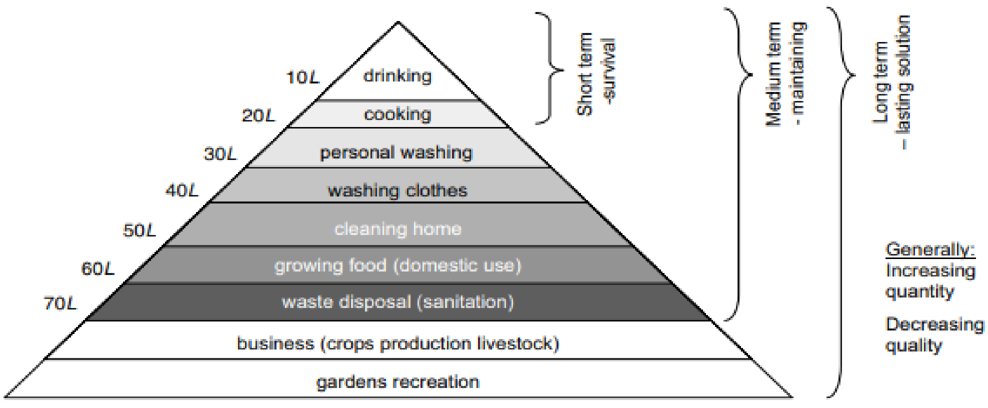

Figure 3, base demand chart (WHO, 2017)

2.2.1 Demand pattern

The demand pattern tests the network's worst-case scenario when water is used simultaneously in larger quantities during the day. A generic daily domestic pattern is pictured in Figure 4, where the demand is larger in the morning and afternoon, whereas it is lower during nighttime. The base demand is multiplied by the multiplier, which causes variations in the demand; therefore, to keep the average base demand accurate, the average value of the multiplier must be equal to one. The multiplier can, during peak hours, reach up to 2.5 times the average demand (Arnalich, 2020).

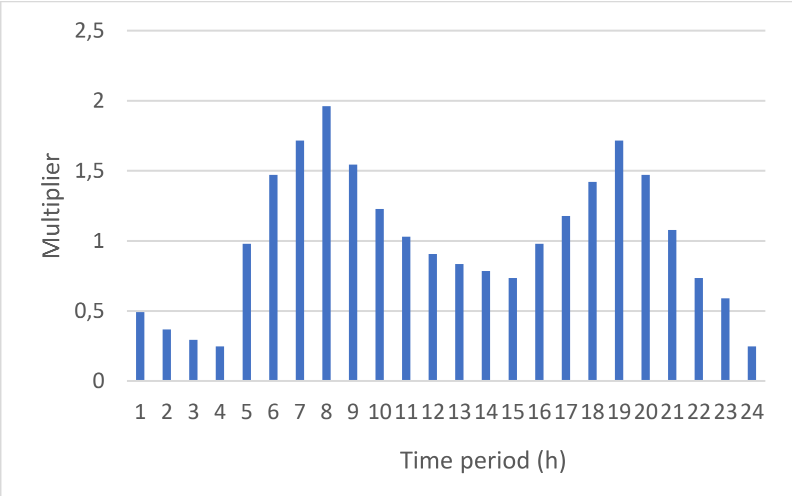

Figure 4, generic domestic water demand pattern, 24h

A theoretical demand pattern where irrigation is included every third day is depicted in Figure 5. The multiplier can in this case reach five times the average base demand during irrigation hours.

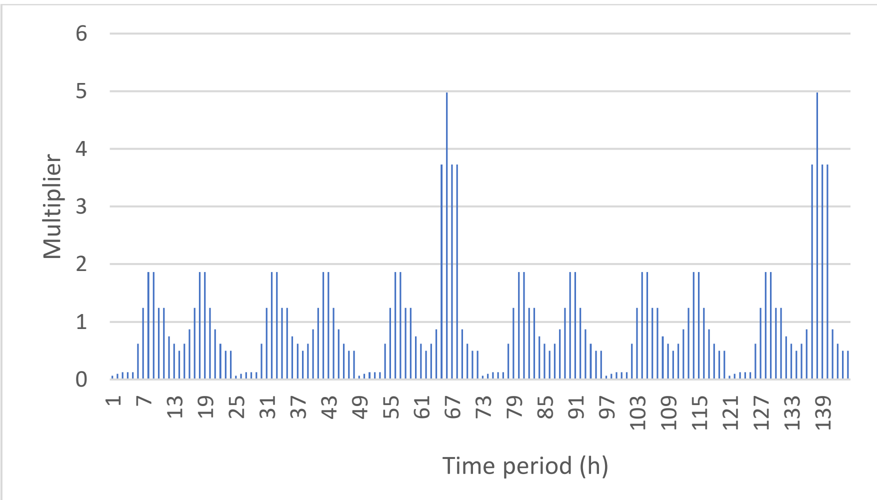

Figure 5, theoretical demand pattern with two irrigation cycles included, 72h

In addition to daily variations of the water demand, weekly and monthly water demand and supply variations occur as well. These can depend on cultural and seasonal changes and varies throughout regions. If no further survey of the weekly and monthly variations is done from the area, a generic multiplier of 1.4 can compensate for the changes.

Unaccounted water uses such as leaks, unauthorized connections, and spilled water when filling containers is about 20% of the average demand; therefore, another multiplier of 1.2 is added to the base demand.

In a worst-case scenario, all the multipliers are added to the base demand simultaneously, which equals 4.2 times the base demand. Therefore, the system should be designed to sustain the worst-case scenario, where the design flow is the sum of all the taps opening simultaneously, which is the base demand multiplied by 4.2 (Arnalich, 2020).

Preferably, surveys need to be done in the area of interest to find a suitable indicative demand and supply patterns. If no data are available, the presented generic numbers can be used.

2.3 Water quality, analyses, and treatment

The purpose and end-use of the water determine whether treatment of the water is needed or not, i.e., if it is for drinking or irrigation. Treating irrigation water may be considered a waste of resources but, depending on the demand ratio of the drinking water and irrigation water and the total flow, the water qualities can be integrated into one system if it is economically more beneficial than building two separate systems. Therefore, if drinking water is included, treatment is a necessity for the network. Untreated water may also cause more wear to the system and clog the pipeline depending on the quality. Although water treatment is essential, large quantities of reasonably good quality water are preferable to an insufficient quantity of good quality water. An inadequate amount of water cannot ensure minimum hygiene levels, potentially leading to more acute health problems (Action contre la faim, 2008).

In many developing countries lacking public distribution system, the cleaning and purification responsibility of raw water is left to the consumers themselves. Common domestic treatment methods are boiling the water to kill bacteria, filtrating the water through ceramic filters, or solar disinfection (SODIS) (Zinn, et al., 2018). Although these methods can be effective treatment methods to obtain drinkable water, it takes time from other activities, such as education and work, if people treat the water themselves. Also, it requires education and knowledge of the dangers of drinking untreated water for people to understand to purify the raw water themselves, which risks the spreading of water-borne diseases. Therefore, large-scale centralized water purification is a significant step to enable inhabitants focus on other tasks than water treatment and to meet the sustainable development goals such as “clean water and sanitation” as well as “decent work and economic growth” amongst others.

The treatment in a centralized cleaning system is often done in multiple steps depending on the raw water quality. The water treatment method and number of steps required are based on the raw-water quality and catchment method (see section 2.4.1). Therefore, the intake raw-water quality must be analyzed before choosing a suitable method.

2.3.1 Common Water contaminants

The water source is either groundwater, surface water, or rainwater. Depending on the source and location of the water, the quality may vary. For instance, the type of soil and rock the water runs through significantly affects contaminants it may contain. Also, depending on the season when the flow and temperature varies may change the water

quality. In addition to the natural contaminants, human activities pollute the water, such as poor agricultural practices, dumping garbage, and open defecation (sswm, 2021).
Water contaminants are divided into three qualities, chemical, biological and physical.

- Chemical
    - Natural organic chemicals and materials
    - Toxic metals and solvents
    - Inorganic substances
- Biological
    - Pathogens and microorganisms such as bacteria, viruses, protozoa, and worms
- Physical
    - Suspended particles (turbidity), temperature, odor, color and taste

The contaminants are furthermore often divided into primary and secondary contaminants. In general, the primary prioritized contaminants are biological pathogens and chemicals known to cause adverse health effects. Non-harmful contaminants that do not cause severe health effects but influence the smell, taste, color, and turbidity of the water or cause technical effects that can damage pipelines or appliances are secondary contaminants such as zinc iron particulate matter and humic material (WHO, 2017).
The World Health Organization has complete lists summarized on most known contaminants, a few of the most common are summarized in Table 2.

Table 2, overview of main pollutants (Action contre la faim, 2008)

Pollution type	Contaminant	Origin	Effect
Chemical: Natural organic components	Easily biodegradable organic matter, polluting if found in high concentrations	Wastewater from domestic use Food and paper industries	Increases the number of bacteria, which reduces oxygen levels in the water and causes organisms and fish to die of asphyxiation
Chemical: Toxic	Heavy metals, e.g., cadmium, lead, mercury… Solvents Pesticides	Insecticides and weed killers Chemical and metal industries	Carcinogenic Can cause various disorders and possibly death
Chemical: Nutritive substances	Nutrients, such as nitrogen, potassium, and sodium. Necessary in small quantities but polluting if found in high concentrations	Domestic wastewater Agriculture, fertilizers Animal excreta	Can cause vegetable overproduction leading to asphyxia of fish
			NO_2 blood disease provoking to babies and fetuses
Biological	Virus Bacteria	Excreta	Transmission of diseases such as Cholera and Rotavirus
Physical pollution	Suspended particles	Soil washing	Non-aesthetic
	Temperature	Industry water cooling	Asphyxiation of fish
	Radioactive elements	Nuclear plant	Disease and mutation of aquatic organisms

2.3.1.1 Biological contaminants

In rural areas, industries are quite limited and the primary source of biological pollution is contact with excrements which spread diseases. Excreta contain a high number of microorganisms, which are not visible to the naked eye. Water sources can easily be polluted in various ways, if they are not properly protected. Especially surface water sources need to be assumed to have been polluted and therefore require treatment. Table 3 presents the most common diseases that can spread through poor water treatment and hygiene standards.

Table 3, common diseases transmitted by excrements (WHO, 2017)

Microorganism	Disease	Symptoms
Virus	Hepatitis A, E	Jaundice (liver)
	Poliomyelitis	Paralysis followed by after-effects
	Typhoid	Nervous and digestive disorders, fever
Bacteria	Shigella	Diarrhea, vomiting
	Cholera	Dysentery
Amoebas	Amoebiasis	Diarrhea, vomiting
Worms	Worm infection	Gastro-intestinal disturbances

Water age is another essential quality to consider when distributing water through a pipeline system. Water stagnation and excessive water age can cause chlorine disinfection byproducts (CDBP) when the disinfectant concentration is not strong enough to control microorganisms. The water age is defined as the time it takes for the water to reach its consumers from the water source and is, therefore, influenced mainly by the distribution system's flows and pipe lengths. Often in dead ends or large tanks, water becomes stagnant and degrades. Satellite chlorination can hinder microorganisms from forming (Ingeduld, 2017). An optimized pipeline with loop topologies can keep the water flowing in the pipes, decreasing the water age — more details in section 2.4.5 about pipeline layout designs.

2.3.1.2 Chemical contaminants

Common natural chemicals found in East African waters are Calcium, Magnesium, Potassium, Sodium, Bicarbonate, Carbonates, Chloride, Nitrate, and Sulphate ions.

Although water can contain many constituents, generally, only a few contaminants propose a health risk. Therefore, different parameters require different priorities for their management. Some propose direct health risks if found in large concentration, while others do not cause severe health effects but cause acceptance problems from the users due to bad odor or taste. Table 4 presents some general chemical contaminants that can cause health effects, and Table 5 presents chemical substances that can cause acceptance problems and wear to the distribution system (WHO, 2017).

Table 4, primary chemical contaminants presenting a sanitary importance to the drinking water (WHO, 2017)

Inorganic substance	Maximum concentration	Origin	Effect
Arsenic	0.01 mg/l	Industrial waste, soil and rocks	Carcinogenic, cardiovascular, neurological and other conditions
Fluoride	1.5 mg/l	Rocks and soil, fish food, manure, industrial pollution	High concentrations and long-term exposure can lead to dental and bone fluorosis
Manganese	0.5 mg/l	Rocks and soil	Toxic effect on nervous system, turbidity and taste
Nitrites	3 mg/l	Fertilization from farms, nearby septic tanks.	Harmful to infants, can cause blue baby syndrome
Nitrates	50 mg/l	Soil, fertilizer, organic substances	Harmful to infants, can cause blue baby syndrome
Chlorine	5 mg/l	Disinfection for water tanks and pipelines, the excess amount can cause symptoms	Eye, nose, and stomach discomfort. Odd taste.

The physical appearance of the water can cause acceptance issues for the users, although the water is safe to drink. Therefore, the contaminants presented in table 6 are regarded as a secondary priority. The color of the water needs to be relatively clear, and the taste and odor should be neutral. Usually, a turbidity of lower than 5 NTU is acceptable for the color. Table 5 summarizes some chemical contaminants that can cause acceptance problems (WHO, 2017).

Table 5, secondary chemicals causing acceptance problems and wear to the pipeline (WHO, 2017)

Inorganic substance	Maximum values	Origin	Effect
Aluminum	0.2 mg/l	Coagulants in water treatment, industries	Bluish discoloration
Ammonia	1.5 mg/l	Wastewater, plants, excreta, organic materials	Taste and odor
Sulphur hydrogen	0.05 mg/l	Rocks, anaerobic organic substances	lethal by inhalation
Chloride	250 mg/l	-	-
Sodium	250 mg/l <	-	Taste problem
Hardness (Calcium, Magnesium)	200 mg/l <	Water-dissolved minerals.	Can cause clogs due to buildup of deposits. Detergents hard to use for dishes and clothes washing.
Phosphate	No standard	Organic substance, detergent, manure	
Potassium	No standard	Manure	
Sulphate	250 mg/l	Rocks, soil, industries	Gastro-intestinal irritation, taste, aggressive to concrete
Iron	0.3 mg/l	Common contamination from soil	Orange stains, metallic taste, bacteria feed on iron which can cause clogging

2.3.2 Analysis methods in the field

Generally, water can be used for consumption if it has no taste, smell, or significant color within the user's acceptance, nor should it have been in contact with excrements or near any other human activity, such as agricultural fields, industries, or garbage. Field measurements, such as EC, pH, turbidity, and TVC, can be conducted to overview the water quality; however, further laboratory analyses are recommended before determining treatment methods due to the numerous possible contaminants. Qualitative field observation of the water's quality is presented in Table 6, however, note that pathogens are not visible to the naked eye (CAWST, 2009).

Table 6, Qualitative observation of the water (CAWST, 2009)

Observation	Possible contaminant
Foamy	Detergents
Black in color	Manganese or bacterial growth
Yellow, brown, or red	Iron
Yellow or dark brown	Tannis and pigment from leaves and back
White deposits or scale	Hardness, dissolved metals
Muddy, fishy, earthy, peaty odor	Algae, bacteria, organic matter
Rotten egg odor	Hydrogen sulfide
Chlorine odor	Chlorine residual from the treatment process
Bitter or metallic taste	pH, zinc, copper

An electro conductivity (EC) meter is a good extensive analysis method to measure the water quality. Conductivity measures the capacity of the water-conducting current between two nodes, therefore indicating how much minerals and salts are present in the water (such as $Ca^{+2}, Mg^{+2}, Na^{+}, K^{+}, Mn^{+2}, Cu^{+2}, Zn^{+2}, Fe^{+3}, Cl^{-}, SO^{-4}$ and NO^{-3}). EC should be measured in 20-25 °C water to follow the standard values as conductivity increases with temperature. Conductivity is usually measured in micro-Siemens by centimeters but can also be measured in total dissolved solids (TDS) multiplied by 0.64. Table 7 describes different values acceptable drinkable standardized values (WHO, 2017).

Table 7, conductivity values (WHO, 2017)

Values µS/cm	Interpretation
µS/cm < 50	Low conductivity, unpolluted rainwater, water deprived of minerals, not recommended for consumption
100 < µS/cm < 500	Average conductivity. Water containing minerals, suitable for consumption
500 < µS/cm < 1000	High conductivity, rich in minerals, possible brackish taste but drinkable
1000 < µS/cm	Very high conductivity. Strong possibility of salinity and taste problems, not suitable for consumption.
2000 = µS/cm	Maximum limit for consumption and agriculture
3000 = µS/cm	Maximum limit for cattle

Total dissolved solids (TDS) or total suspended solids (TSS) measures the turbidity in the water, i.e., the transparency of the water. Suspended particles such as organic waste, organisms, clay, etc., can affect the color, taste and smell, and the health standards of the water. High turbidity can also stock filters, fill tanks and pipes with sand and mud, and damage valves and taps. If chlorination is used, even a low level of turbidity prevents chlorine from killing germs effectively, since micro-organisms can attach themselves to particles. Turbidity is measured in Nephelometric Turbidity Units (NTU); for drinking water, less than 5 NTU is recommended, ideally 0.1 NTU. It can easily be measured in the field with a turbidity tube meter or an electronic turbidity meter. Turbidity tends to be lower during the dry season and higher during the rainy season due to higher flows that stir up the surrounding in the wet season (Action contre la faim, 2008).

Table 8, turbidity values (WHO, 2017)

Values	Interpretation
NTU < 5	Clear and pure water to the naked eye Can be directly filtered and chlorinated
5 < NTU < 30	Slightly colored Must be filtered before chlorination
50 < NTU	Colored Pre-treatment needed before chlorination, e.g. with a slow sand filter (SSF)
200 < NTU	Must be pre-treated before SSF e.g. with sedimentation and filtered before chlorination.

The acidity of the water can be measured on the field with colorimetry or a pH meter. A high pH value can cause a bitter taste, form deposits in the system, and reduce the effectiveness of chlorine disinfection. Low pH can corrode or dissolve metals, causing wear to the distribution system (Swenson & Baldwin, 1965). Table 9 shows the guideline values of suitable pH values for drinkable water.

Table 9, pH values (WHO, 2017)

Values	Interpretation
pH < 5	Strong acidity Possible chemical pollution, rarely in natural waters Not drinkable
5 < pH < 6.5	Average acidity, not drinkable
6.5 < pH < 7	Low acidity, can be corrosive but drinkable
7 < pH < 8	Neutral, found naturally in limestone areas, drinkable
8 < pH < 9	Average alkalinity, drinkable
9 < pH	Strong alkalinity, found in stagnant water with many plants, not drinkable

An effective way to analyze if excreta has polluted the water is by filtering the water sample through a membrane filtration (0.45 μm) to find thermotolerant bacteria. By incubating the filter in a warm and nutritious environment, visible colonies will form on the filter after 16-18 hours, if bacteria are present. About 90% of thermotolerant bacteria is Escherichia Coli (E. coli) which is resistant outside the body and can therefore be found in polluted waters, which indicates that excreta have contaminated the water. Therefore, other microorganisms from excrements can also be present. (Action contre la faim, 2008). For a more comprehensive biological analysis, the Total Viable Count (TVC) is tested in the laboratory to measure the overall number of bacteria.

Table 10, excreta analysis (WHO, 2017)

Type	Values	Interpretation
Thermotolerant coliforms (TTC)	0 / 100 ml	Fecal pollution indicator
Fecal streptococcus	No standard	Fecal pollution indicator
Total Coliforms	0/100 ml in 95% of tests	Indicator of the treatment efficiency

The field analysis methods can give reasonable qualitative indications of which pollutants are present in the water; for further analyses laboratories are required to determine which specific chemical and biological pollutants are present, since each contamination requires its own analysis. Therefore, detailed analyses can be expensive, and only prioritized pollutants and potential elements found from the general field methods need to be analyzed.

2.3.3 Water treatment methods

The treatment methods required depends on the water quality analyses from the raw water source and the intake method (see section 2.4.1). Depending on the water quality results, suitable treatment methods can be selected.

A centralized treatment system is often designed to remove several contaminants, chemicals, pathogens, and suspended solids; therefore, multiple steps are required to treat different qualities of the water. The primary step removes the largest physical particles and turbidity before entering further into the system. The secondary step removes most of the chemical substances and pathogens, and the tertiary step eliminates any remaining disease-causing pathogens and stops new ones from forming in the system. (sswm, 2021). The treatment methods should withstand various flows depending on the seasons (WHO, 2017).

2.3.3.1 Primary step

If direct catchments are used, a physical filter is needed at the inlet to reduce the largest debris entering and stocking the pipeline system. Bar screens or racks with 15-25 mm gaps remove the largest debris; however, they need to be cleaned regularly, especially during the rainy season.

If the water is very turbid (>200 NTU), a sedimentation tank is needed before further treatment. Sedimentation is a physical treatment process where coarser suspended particles heavier than water are removed by settling them down to the bottom of a tank. There are many different types and shapes of sedimentation tanks, whereas the most preferred and widely used are rectangular shaped tanks with horizontal flow, partly because maintenance costs are low for this type. (Anupoju, 2021). Figure 6 depicts the principle of a sedimentation tank. The inlet zone allows uniform water dispersion into the tank's entire cross-sectional area. The settling zone is where the particles settle through the flowing water. Particles accumulate at the sludge zone, from which they need to be removed regularly. The outlet zone collects the clarified liquid uniformly from the cross-sectional area.

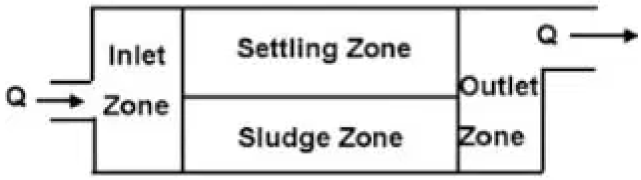

Figure 6, side view of a rectangular sedimentation basin, showing the four typical zones (Anupoju, 2021)

The flow velocity of water must be reduced for the particles to settle down; the velocity at which the particles are settling is called settling velocity (V_S). The settling velocity depends largely on the size of the particles and the relative density of the particles to the fluid. There are two settling types, discrete settling and flocculated settling, illustrated in Figure 7. Due to the larger flocculation size, they settle down more rapidly than discrete particles. If necessary, flocculants can be added to the fluid to flocculate discrete particles (Wisniewski, 2012).

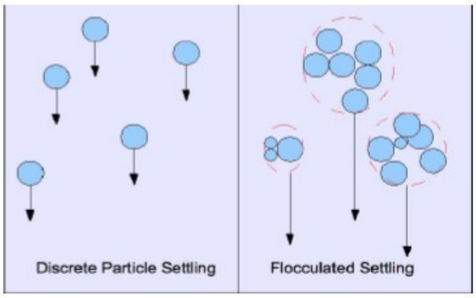

Figure 7, difference between discrete and flocculated settling, (Wisniewski, 2012)

Table 11 gives an overview of the usual settling velocity for respective particle sizes.

Table 11, an overview of settling velocity of respective particle sizes (Pilgrim, 2001)

Classification of particle size	Particle diameter (µm) d	Settling velocity (m/s) V_s
Very coarse sand	2000	0.2
Coarse sand	1000	0.1
Medium sand	500	0.053
Fine sand	250	0.026
Very fine sand	125	0.011
Coarse silt	62	0.0026
Medium silt	31	0.0066
Fine silt	16	0.0018
Very fine silt	8	0.0004
Clay	4	0.00011

The time it takes for the particle to settle is called hydraulic retention time (HRT). The HRT can be calculated by dividing the height (h) of the basin by the settling flow of the particle (V_s), as described in equation 7.

$$HRT = \frac{h}{V_s} \tag{7}$$

Rough dimensions for the sedimentation tank can be calculated by multiplying the design flow ($\dot{Q}$) with the retention time, as equation 8. The height (h) of a sedimentation tank is typically between 2 and 3 meters, and the length (l) is four times the width (w), as equation 9. Preferably, the water flow velocity should be less than 0.005 m/s to avoid turbulence in the water which may disrupt the sedimentation process.

$$V = \dot{Q} \cdot HRT \tag{8}$$

$$V = h \cdot w \cdot l = h \ \cdot 5w \tag{9}$$

To improve the efficiency of a sedimentation basin inclined plate settlers (IPS-design) can be added, reducing the area needed for the sedimentation process to occur. This design is suitable for hilly areas where the construction of large facilities might be challenging (Wisniewski, 2012).

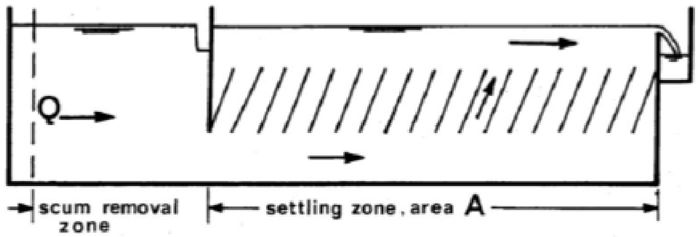

Figure 8, Sedimentation tank with an Inclined Plate Settler - design (Wisniewski, 2012)

2.3.3.2 Secondary step

The secondary treatment step removes the bacteriological pathogens from the water, making it safe to drink. The secondary step can use slow sand filters as a treatment method or membrane filters, if the water is severely contaminated.

Slow sand filters (SSF) use gravity to let water flow through a filter of sand (with an adequate particle diameter size of 0.15-0.3mm). On the top few centimeters, a layer on the sand forms a biological layer known as the "schmutzdecke," which allows the filter to remove turbidity and microorganisms from the water. A well-designed SSF can effectively remove biological pathogens and turbidity in a single step. The treated water is collected from underdrains at the bottom of the filter. The top sand layer accumulates solids and needs to be replaced periodically. The typical flow rate is usually between 0.1 and 0.3 m^3/h per square meter (WHO, 2017).

SSFs are adequate for low turbidity water. If the water source has a high degree of turbidity (>200 NTU) or algae contamination it can cause the filter to clog rapidly, in which case pre-treatment of the water is needed, e.g., sedimentation (see section 2.3.3.1).

SSFs are highly reliable, simple to construct, and have low lifecycle costs. The construction and maintenance only require basic skills, making this treatment method a promising filtration method in rural communities with reasonably good initial surface water source quality (sswm, 2021).

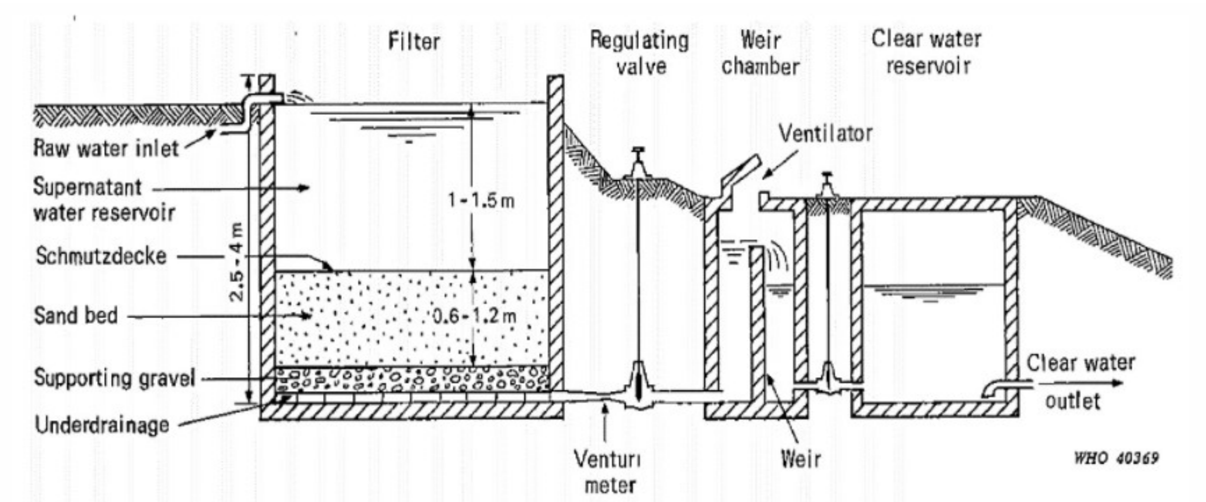

Figure 9, illustration of an SSF construction (Huisman & Wood, 1974)

SSFs are very efficient for removing bacteria and viruses; with an efficiency ratio of 90-99% removal rate, SSFs will seldom indicate microbial contamination present in the outlet. However, SSFs are not effective for removing most chemicals, as shown in Table 12 (WHO, 2017).

Table 12, an overview of SSF filtration efficiency (sswm, 2021)

Effective filtration ratio	Somewhat effective filtration ratio	Not effective filtration ratio
Bacteria	Odor, taste	Salts
Protozoa	Fe, Mn	Fluoride
Viruses	Organic matter	Trihalomethane (THM)
Heavy metals (Zn, Cu, Cd, Pb)	Arsenic	Majority of chemicals
Turbidity	-	-

Disinfectants, such as chlorination, are usually added after the SSF to remove the remaining contaminants (see section 2.3.3.3), decreasing any activity for bacteria during the storage and distribution of the water. If chlorine is introduced, it must be added after the SSF not to interfere with the schmutszdecke (sswm, 2021).

If the raw water contains contaminants that SSFs are ineffective for, membrane filters can be used, although these filters need more efficient pre-treatment than SSFs not to clog the filter. Membrane filtration uses a thin, porous sheet to separate contaminants from the water when a force is applied through the filter. The membrane process can remove bacteria, microorganisms, particulate material, micropollutants, natural organic material, and chemicals such as arsenic and fluoride. The membrane filtration can produce potable water from ground, surface, and seawater sources. However, it is expensive, and the membrane can be difficult to obtain in rural developing areas, also maintenance and service of the membrane filters require professional expertise which can be lacking in remote areas (sswm, 2021).

Due to the importation of membrane filters to rural areas and continuous expert maintenance needed, membrane filters are not recommended to use in simple distribution systems and, thus, are not further presented in this context.

2.3.3.3 Tertiary step

The tertiary step is a treatment within the distribution system that prevents recontamination and forming of microorganisms in the pipelines and water tanks, therefore ensuring the drinking water quality throughout the system.

Chlorination effectively kills remaining pathogens and hinders algae, fungi, and bacterial growth in pipes and tanks. However, if chlorination is misused, by-products can form that are toxic and cause odor and taste problems to the drinking water (sswm, 2021).

Different types of chlorine can be used which are in gaseous form, liquid form, or powder form. Chlorine in gaseous form is usually used in public systems due to higher concentrations; however, it can be dangerous if used by untrained personnel. Smaller systems or private households use chlorine in liquid or powder form.

The presence of chlorine residuals in the water after treatment indicates that all oxidable substances in the water have been oxidized and, therefore, any biological activity disabled. By maintaining a low chlorine residual concentration assures a successful treatment; therefore, the concentration should be constantly measured throughout the distribution system to prevent recontamination in the pipelines and tanks. An ideal chlorine residual ratio is 0.2-0.5 mg/l. If biological contamination spreads in the system, shock chlorination can be needed with a 50 mg/l chlorine residual concentration. Otherwise, an excessive chlorination is corrosive to equipment and undesirable to consumers due to taste and smell (Oram, 2021).

The amount of disinfectant needed depends on the concentration of the chlorine type and the initial water quality. The time required for the disinfectant to react is called Contact time (*CT*). *CT* depends on the temperature, pH, and targeted microorganisms. It can roughly be estimated by a K-factor that is temperature and pH-dependent, whereafter it is divided by the targeted chlorine residual (Oram, 2021).

Table 13, K-factor (Oram, 2021)

Highest pH	Water temperature for *K* factors		
	>10 °C	7 °C	< 4 °C
6.5	4	5	6
7	8	10	12
7.5	12	15	18
8	16	30	24
8.5	20	25	30
9	24	30	36

$$CT = \frac{K}{C} \tag{10}$$

CT is the contact time in minutes, *K* is the factor from Table 13, and *C* is the chlorine residual concentration, usually between 0.3 and 0.5 mg/l.

Chlorination disinfection by-products (CDBPs) can be produced in the chlorination process (see Table 14) if the water contains natural organic matters such as fulvic and humic acids, usually found in tropical regions surface water sources. If the water is very turbid (>20 NTU), pathogens can attach to suspended particles, reducing the effectiveness of the chlorine treatment. For these reasons, the water should be pre-treated before chlorination is introduced unless the quality of the raw water can be ensured. Pre-treatment also reduces the *CT*, increasing the effectiveness and reducing the amount needed (sswm, 2021).

Table 14, Chlorination Disinfection By-Products

Chlorination disinfection by-products	Effect
Trihalomethanes (THMs)	Carcinogenic, chronic exposure can cause liver and kidney damage
Haloacetic acids (HAAs),	Irritant, corrosive and destructive to membranes
Chlorophenols	Cause taste and odor problems, toxic if found in higher concentrations
Aldehydes, nitriles, chloramines	Taste and odor

2.4 Components of a gravity-fed distribution system

The components in a gravity-fed distribution system are illustrated in Figure 10. The water source and catchment may vary depending on the accessibility and circumstances, whether from a spring catchment, well, borehole, or surface water. The water source and its quality determine the treatment method required for the system. The first tank in the pipeline system is a header tank used as an initial flow buffer to the rest of the system. If treatment is required, it can also be used as a sedimentation tank and be combined with a slow sand filter. If the elevation difference is too significant and the pressure in the pipeline becomes substantial, break-pressure tanks can be installed as primitive substitutes to pressure-reducing valves to reduce the internal pressure to atmospheric pressure in the pipeline. Storage tanks are the largest components in the system and are used to secure a constant supply regardless of the demand and supply variations. The outlets distribute the water close to the consumers and need to follow certain criteria to be practical.

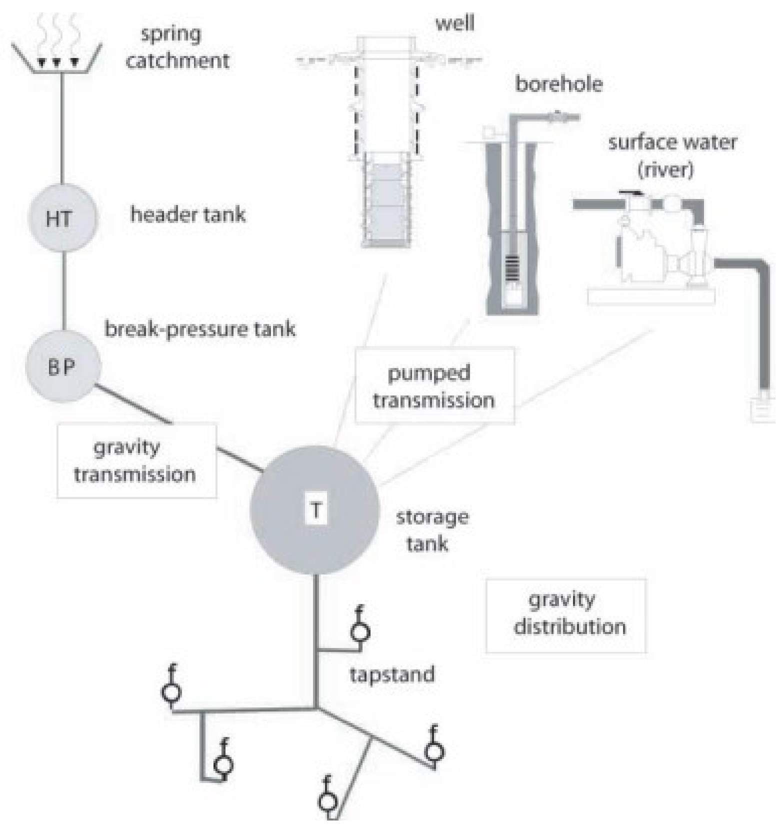

Figure 10, component overview of a gravity-fed distribution system (Sampers, 2005)

2.4.1 **Catchment**

There are generally three different ways of catching water to a distribution system. These are rainwater, surface water and groundwater.

Rainwater can be harvested from rooftops and other large surfaces and is brought by gutters to a water reservoir. Rainwater is mainly a temporal and complementary way of collecting water. Whether it is in an area where it rains a lot or where it is dry and any means of collecting water is used, the method requires large storage volumes to be useful over longer time periods. During long storage periods, the water is easily exposed to contamination. It may also easily become polluted through the catchment process from rusty rooftops or animal excretes. Due to the lack of minerals, the water is acidic and corrosive towards metals (Action contre la faim, 2008).

Surface water is an easily accessible resource to use and can be used in various ways. However, surface water is vulnerable to pollution and may dry out during the dry season. Direct river catchments need a screening of debris to hinder the largest particles to enter the pipeline. A simple way is to build rocks around the pipe which work as a rough filter as well as physical protection (see Figure 11). The intake should at least be more than 30 cm under the surface to avoid floating particles, such as excrements and organic material and algae, to enter the pipeline. Nor should the intake be near the bottom due to risk of sedimented contaminants entering the intake. (Arnalich, 2020) Furthermore, the intake can be equipped with bar screens or racks with 15-25 mm gaps to filter out sediments. To reduce further contamination of the water the direct river intake should be placed upstream of all other activities, such as agricultures, industries and storm water outlets (Action contre la faim, 2008).

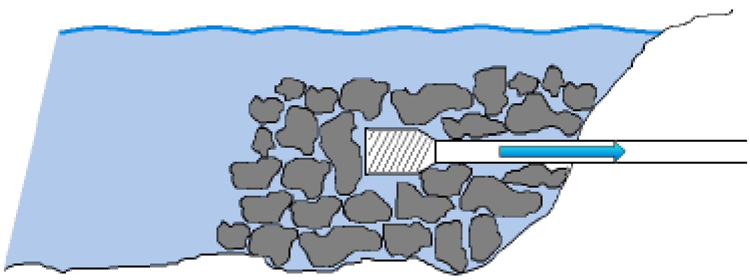

Figure 11, direct river intake with rough screening filter (Arnalich, 2020)

To ensure sufficient flow from direct river intakes small dams can be built to increase the water depth. A dam also creates an area of the water at rest, allowing particles to sediment if it is large enough (see section 2.3.3.1). Dams should be carefully designed, as erosion can easily cause the dam to collapse; moreover, the dam should not flood the area when it is full. Figure 12 illustrates an example of a partially closed river dam. The walls and sides must be reinforced with stones and masonry as not to let them erode. The design of dams varies and depends largely on the specific area it is built on; preferably, it should be built on bedrock in narrow stretches where there is a significant change in the elevation (Arnalich, 2020).

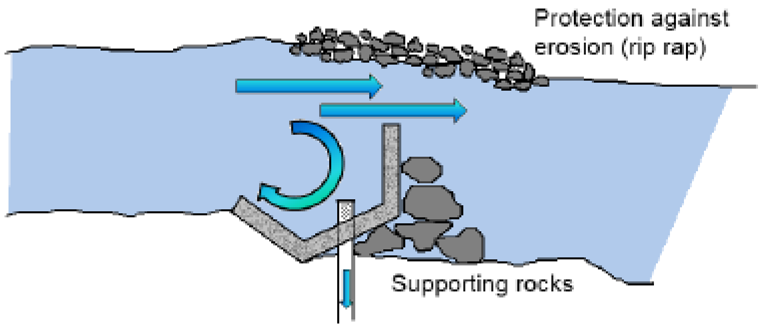

Figure 12, partially closed dam, top view (Arnalich, 2020)

Indirect river catchments are the intake of choice from surface water sources due to them greatly reducing the need for subsequent treatment. Figure 13 illustrates the principle of the method. In a similar way as slow sand filters treat the water, the catchment pipe is buried at least one meter in layers of sand with a progressive reducing diameter starting from 10 mm gravel around the pipe, which significantly reduces the bacteriological contamination from the surface water. In gravitationally fed systems, the underlying pipe can be placed in parallel with the stream flow to increase the catchment area and supply (Arnalich, 2020).

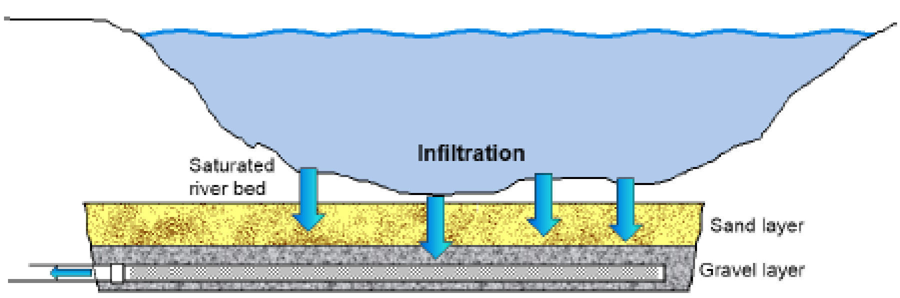

Figure 13, indirect river catchment, side view (Arnalich, 2020)

Water within three meters of the surface is considered surface water, below three meters is groundwater. Groundwater usually has the best quality but may be the most expensive option to utilize. It tends to have low bacteriological contamination due to the natural cleaning occurring through the flow through the ground. However, there may be a higher concentration of minerals in the water depending on the soil, which must be analyzed (Action contre la faim, 2008).

Spring catchments in slopes work as great intakes for gravity-fed systems. The flow is generally more reliable and stable throughout the seasons than surface water sources. The spring catchment intakes are very simple and do not require advances materials to construct, as shown in Figure 14. The water source is protected from probable contamination with an impermeable layer of clay before the intake; additionally, the area can be fenced off to stop animals from entering the collection zone. The underground water flow is collected with a concrete wall and a collection bowl which is pressurized by the slope. If possible, a header tank can be combined with the intake (Arnalich, 2020).

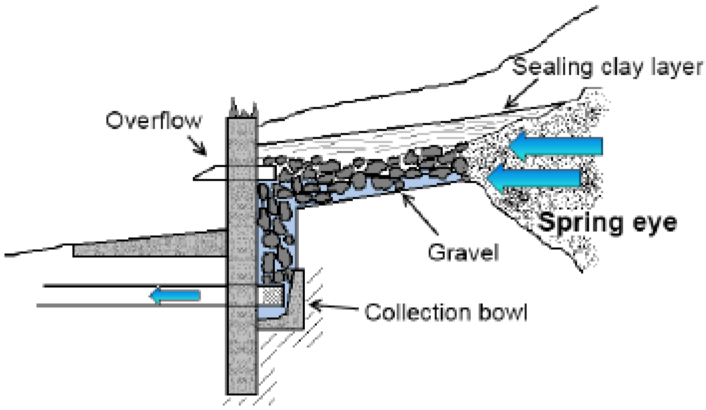

Figure 14, spring catchment (Arnalich, 2020)

Artesian springs are natural wells containing groundwater where two impermeable layers of soil or rock have an opening in between, and the hydrostatic pressure of the underground river is positive in the opening. Artesian springs are very easy to utilize; a concrete box with a lid on can be built directly on top of the spring where the walls are sealed with the lowest impermeable layer, and the bottom is covered with gravel for the water to flow through. Usually pumping power is required if artesian springs are utilized in a larger distribution system (USGS, 2021).

Wells and boreholes can be dug where there are known deeper groundwater sources, ranging for wells 3-30 meters deep for wells, respectively for boreholes up to 60 meters deep. The construction for wells and boreholes can be expensive, especially boreholes, since drilling equipment is needed. The area around the intakes needs to be protected with impermeable layers as not to let contaminations soak through the soil. Both methods require pumping power to extract the water (Action contre la faim, 2008).

A summary of the different catchment methods is provided in Table 15.

Table 15, overview of catchment methods (Action contre la faim, 2008)

Water resource	Supply method	Advantages	Disadvantages
Rainwater	Rooftop harvesting	- Good quality if collected under good circumstances	- Complementary or temporary resource - Can be easily polluted - Lacking minerals
Surface water	Direct intake	- Rapid and easy to implement	- Generally, requires a treatment unit due to high probability of fecal pollution - Can be very turbid
	Indirect intake	- Clear water and good bacteriological quality	- Low-lying position may cause insufficient pressure to the system - Difficult to work in flooded areas
Groundwater, > 3 meters below the surface	Well	- Clear water and good bacteriological quality	- Easily polluted if badly protected - Requires lots of work to construct - Requires pumping power
	Borehole	- Clear water and good quality - Deep water catchment	- Construction cost very high - Requires pumping power
	Spring catchment	- Clear water and good quality	- Flow may not be sufficient

Some catchments require pumping power to extract the water. If they are integrated into the distribution system, an electric pump is needed. The rough size of the pump is determined by calculating equation 11 (Engineering ToolBox, 2021):

$$P_h = \frac{q \cdot p}{3.6 \cdot 10^6} \tag{11}$$

where P_h is the hydraulic power (kW), q is the required demand flow (m^3/h) and p is the pressure (Pa). To determine a more accurate and dynamic use of the pump the pump pressure curve and pipe curve should be considered, as well as the pipelines infrastructure, if there is a tank downstream that the pump need to smoothen out during peak demands.

2.4.2 Header tank

The header tank is placed at the beginning of the pipeline and works as a buffer to the flow variations from the water source to stabilize the flow. Since the header tank is the first tank in the network after the intake, it also determines the static and dynamic piezometric lines for the rest of the line. It can also work as a primary treatment for the water and be dimensioned as a sedimentation basin. If further treatment is needed, the header tank should be combined with an SSF. See section 2.3.3.1 for dimensioning the header tank as a sedimentation tank and section 2.3.3.2 for dimensioning an SSF.

2.4.3 Brake pressure tank

A brake pressure tank (BPT) can be installed if the pressure in the network between the source or storage tank and the outlet exceeds 30 meters. The BPT reduces the pressure in the lines to atmospheric pressure. BPTs are more commonly used in developing regions as substitutes for pressure-reducing valves due to their simplicity and availability. There is no minimum size for a BPT, but generally, a volume of $1-2\ \text{m}^3$ allows for less abrupt flow and, therefore, longer life expectancy of the float valve (Arnalich, 2020).

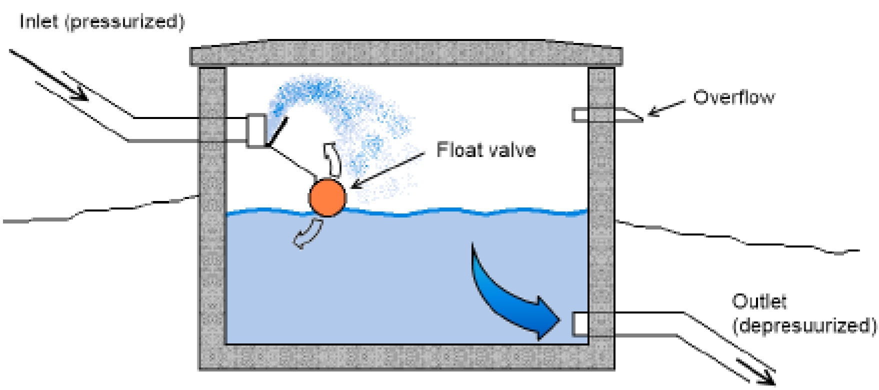

Figure 15, brake pressure tank illustration (Arnalich, 2020)

2.4.4 **Storage tank**

A storage tank is not necessarily needed if the maximum flow demand is smaller than the inflow from the water source. However, if a storage tank is needed, it needs to be carefully designed. A storage tank should preferably be located on a hill near the largest concentration of outlets and demands.

A rough estimation of the required storage tank size can be calculated by the difference of the largest consecutive inflow during a selected period and the maximum outflow during the same time period, e.g., during a day. Preferably, the time period should also consider different seasons, if possible. The larger the timeframe, the better the estimation; however, a broader timeframe requires more data and research of the local area's water behavior to estimate the pattern (see section 2.2.1Demand pattern) (Arnalich, 2020).

An example of a dimension calculation is presented below. The water tank is fed with a constant supply of water, 1900 l/h. A base demand of 0.5 l/s is multiplied according to the pattern in Figure 4. The balance column calculates the difference for each hour by subtracting the demand from the supply. The water stock is the sum of its previous value added with the current balance value. The necessary water stock is the minimum volume of water per hour in the tank, i.e., the water stock added with its minimum value (Action contre la faim, 2008).

Table 16, storage tank example data

Hour	Constant water supply (l/h)	Multiplier	Water demand (l/h)	Balance (supply-demand) (l)	Water stock (l)	Necessary water stock (l)
1	1900	0.490296	882.5	1017.5	1017.5	1493.4
2	1900	0.367722	661.9	1238.1	2255.6	2731.5
3	1900	0.294178	529.5	1370.5	3626.	4101.9
4	1900	0.245148	441.3	1458.7	5084.8	5560.7
5	1900	0.980592	1765.1	134.9	5219.7	5695.6
6	1900	1.470889	2647.6	-747.6	4472.1	4948.0
7	1900	1.716037	3088.9	-1188.9	3283.3	3759.1
8	1900	1.961185	3530.1	-1630.1	1653.1	2129.0
9	1900	1.544433	2779.9	-879.9	773.1	1249.0
10	1900	1.225741	2206.3	-306.3	466.8	942.7
11	1900	1.029622	1853.3	46.9	513.5	989.4
12	1900	0.907048	1632.7	267.3	780.8	1256.7
13	1900	0.833504	1500.3	399.7	1180.5	1656.4
14	1900	0.784474	1412.1	487.9	1668.4	2144.3
15	1900	0.735444	1323.8	576.2	2244.6	2720.5
16	1900	0.980592	1765.1	134.9	2379.6	2855.5
17	1900	1.176711	2118.1	-218.1	2161.5	2637.4
18	1900	1.421859	2559.3	-659.3	1502.2	1978.0
19	1900	1.716037	3088.9	-1188.9	313.3	789.2
20	1900	1.470889	2647.6	-747.6	-434.3	41.6
21	1900	1.078652	1941.6	-41.6	-475.9	0
22	1900	0.735444	1323.8	576.2	100.3	576.2
23	1900	0.588355	1059.0	840.9	941.3	1417.2
24	1900	0.245148	441.3	1458.7	2400.0	2875.9
TOTAL	45600	-	43 200	-	-	-

The storage tank's minimum capacity (V_{STmin}) is found by subtracting the largest volume ($\dot{V}_{max}$) in the water stock column with the smallest value ($\dot{V}_{min}$), the highlighted values in Table 16.

$$V_{STmin} = \dot{V}_{max} - -\dot{V}_{min} \tag{12}$$

$$V_{STmin} = 5219.7\ l - -475.9l = 5695.6l \approx 5.7m^3$$

The largest water stock difference during the selected time frame is depicted in Figure 16, which gives the minimum variation of the water volume needed during a day.

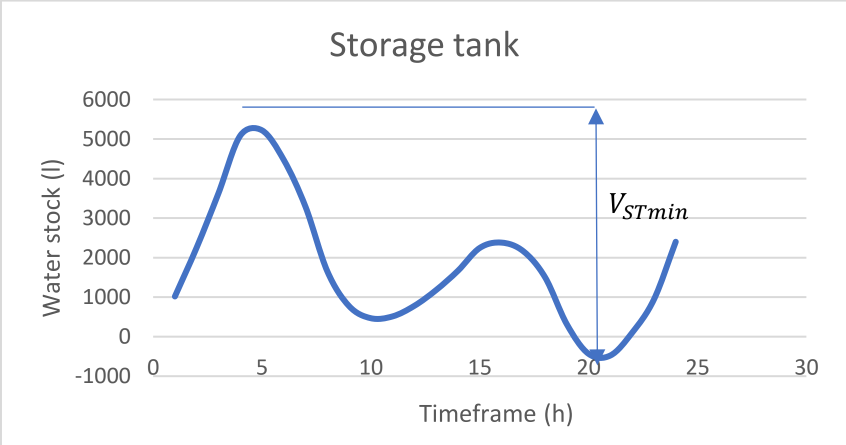

Figure 16, storage tank minimum variation volume during a day

Since the supply is larger than the demand, there will be an overflow in the tank during the selected time period. Therefore, to preserve some of the overflows the daily supply ($V_{tot.sup}$) is subtracted from the average daily demand ($V_{tot.dem}$) to obtain the extra volume (V_{STreal}) needed to hinder a daily overflow, as equation 13 (Action contre la faim, 2008).

$$V_{STreal} = V_{STmin} + (V_{tot.sup} - V_{tot.dem}) \tag{13}$$

$$V_{STreal} = 5.7\ m^3 + (4.56\ m^3 - 4.32\ m^3) = 8.1\ m^3$$

This method of dimensioning works if the water supply and consumption are relatively the same. If the supply is lower than the demand, then an additional intake or an intake with a larger capacity is needed. Alternatively, if the difference in demand and supply is huge, then a more extended time period of the demand and supply pattern must be selected.

2.4.5 **Pipes**

To determine the diameter (d) size of the pipe the topographic profile is analyzed (see section 2.1) . The hydraulic gradient line needs to be above the one bar limit at all times. A larger pipe diameter lowers the liquid's velocity (v); respectively, a smaller diameter increases the velocity and therefore increases the resistance. The design flow (Q) is based on the worst-case scenario demand when dimensioning the pipe diameter, as equation 14.

$$Q = \frac{v}{\pi \cdot \left(\frac{d}{2}\right)^2} \tag{14}$$

Decreasing the diameter increases the headloss and conversely, increasing the diameter decreases the headloss according to the Hazen-Williams formula (see section 2.1.3). If the headloss is larger than the elevation difference of the pipes' end points, the dynamic pressure will be insufficient to transport water to the end point (see Figure 2). For economic reasons, it strives to find the smallest possible diameter for the pipe without breaking the hydrostatic rules; however, smaller than 63mm for network applications is not recommended (Arnalich, 2020).

The maximum pressure in the network is the highest static pressure (see section 2.1.1) that is found in the pipeline, whereafter suitable pipe material and pipe tolerance are chosen.

Table 17, pipe material and pressure examples (Action contre la faim, 2008)

Pipe type	Nominal Pressure (bar)	Maximum pressure, p_{Sta}, (mWG)
PVC pr PE	6	60 m
	10	100 m
	16	160 m
Galvanized iron (GI)	16	160 m
	25	250 m

The choice of the pipe route can be roughly estimated from maps and satellite images; however, the definitive route must be determined on-site with a local who knows the area well and takes into account the following aspects (WHO, 2017):

- Minimize crossings, such as rivers and roads
- Avoid rocky zones, as it is easier for trench digging
- Prioritize accessible zones, e.g., avoid steep slopes, follow existing paths such as roads to ease construction
- Land ownership and authorization

Furthermore, the design of the layout and connections can be varied. Branch topologies are most common in gravity-fed systems; however, if one branch breaks, it compromises the whole distribution to the rest of the network. Another drawback is stagnation of the water, which can be a problem if the water is not constantly used in branched networks. Loop networks are more reliable, since water is distributed through several pipes to the nodes, but this method requires more pipes and is therefore also more expensive. The water circulates more in loop networks reducing the stagnation of the water. In a gravitationally fed system, the loop network would need additional pipes to work properly, as shown in Figure 17. Therefore, whether the advantages for a loop-topology in a gravitationally fed network are more beneficial than the cost to build it must be individually analyzed for each case (Sampers, 2005).

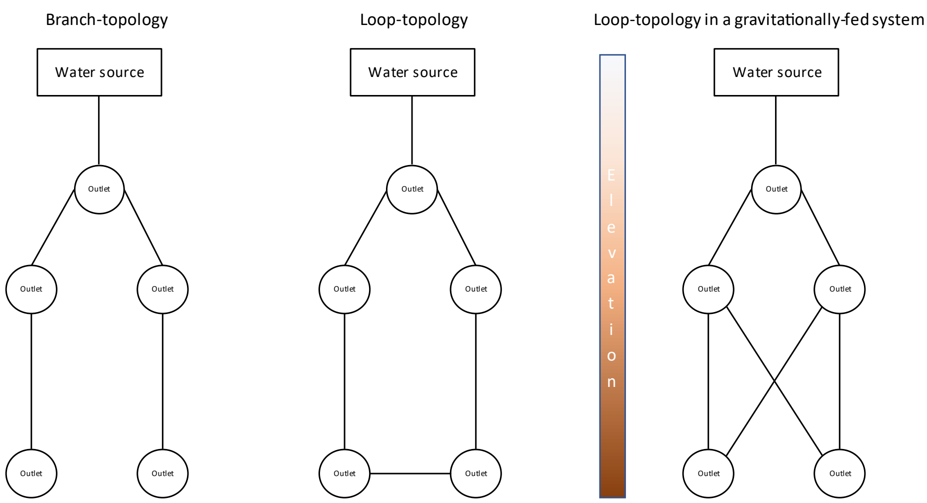

Figure 17, pipeline topology designs

2.4.6 **Outlets**

For practical purposes, the pressure in the outlets of the pipe should be between 1 bar and 3 bar. Less than 1 bar is insufficient to distribute the water, and more than 3 bar can cause problems with opening the outlet valves; thus, when filling the buckets, large amounts of water will splash and go to waste.

It is recommended to have at least one outlet within 250 meters or 4-30 minutes walking distance. Less than a four-minute walking distance might cause overuse by the closest households and, therefore, inequalities between locals. More than 30 minutes of walking distance will defeat the purpose of an easily accessible piped water network. Locals will find other solutions to provide water to themselves, especially if the network is designed to provide more than 50 liters per capita per day.

It is essential for the system to be equally accessible for all users, or else illegal connections to private households may be constructed, which can disrupt the flow to the rest of the users. Satellite images can be used to locate the households if no other data is available (Arnalich, 2020).

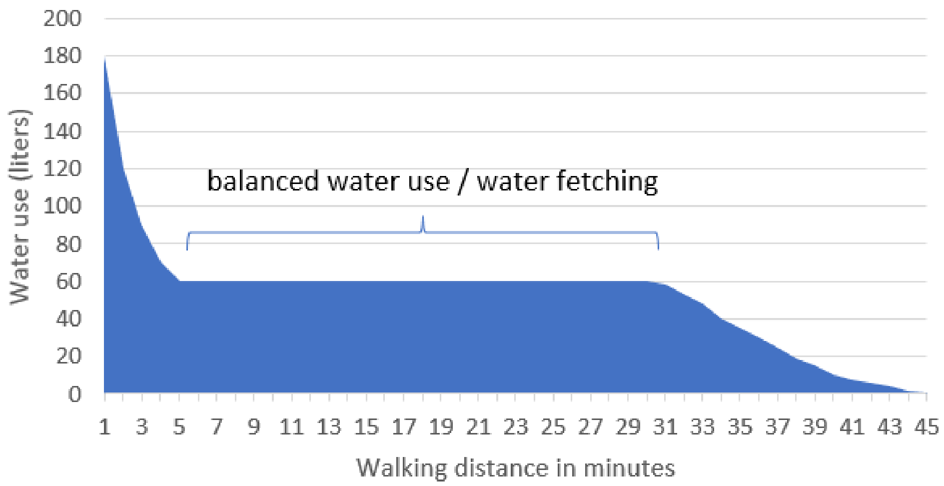

Figure 18, optimal distances between users and outlets (Arnalich, 2020)

3 The water supply situation in Ward 10

3.1 Problem description of the water supply system in Ward 10

According to the 2017 survey, 88% of the inhabitants on Ward 10 have complained that the water access is insufficient, whereas only 26% have access to water from the existing pipeline network, and the rest use wells or other means of collecting water. The average distance from a drinking source is 739 m, ranging from 3 m to 10 km. (Awais, et al., 2017)
The current public water system was built in 2003 by Plan International in cooperation with ZINWA. The primary water source of the existing public system is the Mupenga River 1196m above sea level, which is relatively small and very dependent on the year's season. During the dry season in the spring and autumn, the water flows partially underground, and the surface water is very shallow. During the rainy season in the winter (Dec-March), the water flows are strong (Awais, et al., 2017).
By analyzing the existing system, a few fundamental technical problems have been found. It does not cover the whole area of Ward 10, nor is the single intake from the Mupenga River sufficient to meet the demand of the area that it covers. The measured flow of the Mupenga River in the dry season is 0.91 l/s, whereas the base demand that the area the network covers is 1.27 l/s for 80 liters per capita per day. Furthermore, the current water treatment system in Ward 10 lacks numerous aspects of treatment. It mainly relies on the consumer to treat the water by boiling it or using clay filters to remove pathogens from the water. Boiling the water can take from 30 minutes up to 60 minutes, and only 8% of the inhabitants treat their drinking water. Due to lack of knowledge and education, about 90% of the inhabitants drink the water raw without treating it, even if the water has been polluted by derelict tanks with collapsed asbestos roofs (Chikuhwa, 2021), (Awais, et al., 2017).

Currently, the only treatment in the water supply system in Ward 10 consists of an under-dimensioned physical filter and an insufficient natural sedimentation basin. The water source used in the system is the Mupenga River, a stream draining the rainwater from the mountain ranges. Before the intake of the water distribution system is a natural sedimentation basin. However, the sedimentation basin is too small to work as intended, so excess sediments and mud must be manually removed three times a week during the rainy season and once a week during the dry season, which causes troubles to the water system and maintenance. At the sedimentation basin are two intakes to the pipeline distribution system, a 90 mm pipe and a 110 mm pipe. Each has a physical filter that is poorly constructed; due to too large holes in the filter, numerous smaller objects can easily enter the pipelines. After 1.3 km from the intake are two pressure breaker tanks for the respective pipelines, which helps the sedimentation for some of the remaining suspended substances (Awais, et al., 2017).

Figure 19, Raw water source at Mupenga River (Awais, et al., 2017)

No in-depth water quality analysis has been made in Ward 10 other than a basic analysis presented in Table 18; it should be noted that no biological nor radiological contaminants have been checked during the analysis. Four samples from four different locations were taken and analyzed in 2017, one from a groundwater source, one from a borehole, one from a clinic’s water tank, and one from the Turumhu River. The samples results are compared to the WHO standards for potable water, and it can be noted that most of the samples are well within the range of the WHO threshold value, except the fluoride levels.

Table 18, water quality analysis in Ward 10

Location	Date	pH	Bicarbonate HCO_3 (mg/l)	Conductivity (µS/cm)	Fluorides (mg/l)	Nitrates NO_3 (mg/l)	Phosphorus PO_4 (µg/l)	Sulphates SO_4^{2-} (mg/l)
WHO standards for potable water	2017	6.5-8.5	-	< 400	< 1.5	< 50	< 300	< 500
Ground water source (Patsime)	04.04.2017	6.618	72.844	156.7	2.59	n.d.	68.314	13.510
Samaringa Primary school (Borehole)	04.04.2017	6.808	101.761	232.0	7.53	n.d.	11.037	59.520
Honde Mission Clinic (Tank)	04.04.2017	9.028	41.851	64.4	n.d.	n.d.	19.134	n.d.
Turumhu river	04.04.2017	8.869	57.958	101.6	n.d.	n.d.	0.525	n.d.

The high levels of fluoride may be due to the type of soil in the area the water absorbs. Using surface water as a water source to the system may decrease the fluoride levels in the drinking water; however, surface water is more exposed to bacteriological contamination. According to the World Health Organization, chemical standards can be seen as a secondary concern compared to bacterial contamination (WHO, 2017). The water in the area has been drunk for generations, and no excessive dental problems have been noted due to the high fluoride levels (Mukome, 2021).

Due to the placement of the water intake and lack of nearby households and factories, it is assumed that only physical particulates and natural biological pathogens of various kinds are present in the open water source. No illnesses caused by heavy metals or radiations have been noticed. However, collapsed water tanks that are made of hazardous materials such as asbestos may pollute the distribution lines. In order to clear this issue, whether this is drinkable water, further testing needs to be done.

Some years after the construction of the pipeline system, it had a transfer of responsibility of the management and maintenance due to problems with the water bill payments, and the responsibility shifted from ZINWA to the local Samaringa community committee. After that, the water supply has been free of charge, and maintenance has been carried out by volunteers, whereafter it has been reported that it is leaking and water tanks are deteriorating; however, further data are lacking concerning what exact condition the pipeline is currently in.

Due to lack of water availability from the public pipeline system and mismanagement, the inhabitants of Ward 10 have begun to make their own rudimentary dams with sandbags in nearby streams and collect water with a pipeline system of their own, as can be seen in Figure 20-22. These systems are disorganized and haphazardous, which makes the management of the water resources and availability more challenging, thus draining the rivers more than needed. Some private pipelines are not maintained and may be left to pollute the environment after serving their use (Chikuhwa, 2021).

Figure 20, Rudimentary dam built with sandbags (Chikuhwa, 2021)

Figure 21, lack of management and maintenance leaves the condition of the individual pipelines to the respective users' responsibility (Chikuhwa, 2021)

Figure 22, A section of individual pipeline connections (Chikuhwa, 2021)

3.2 Mapping of the existing network and Ward 10

Through interviews with locals, a digitalized representation of the existing public pipeline was drawn in 2017; however, the illustration (Figure 23) is not to scale nor georeferenced.

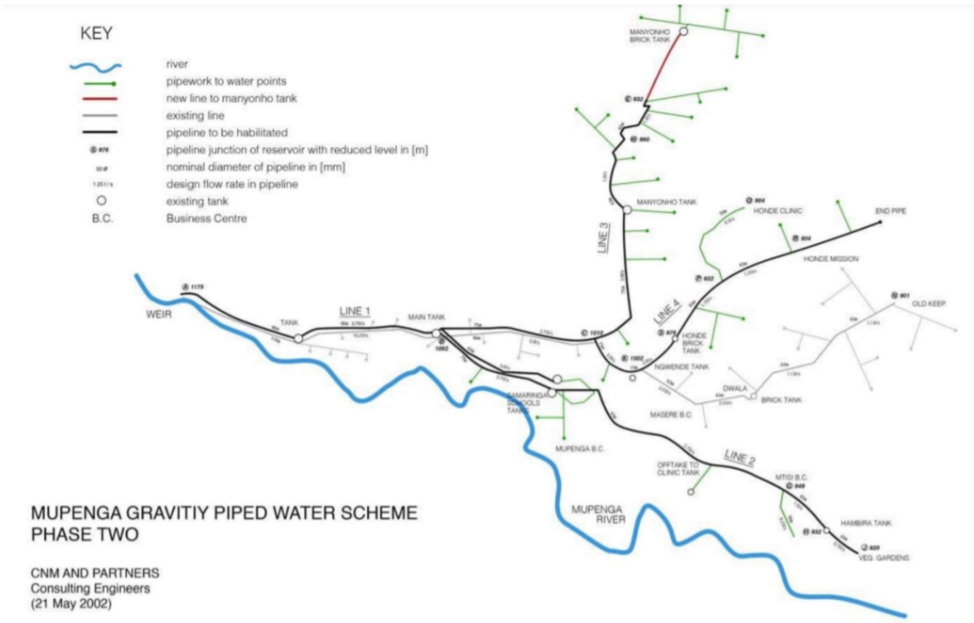

Figure 23, digitalized version of the existing pipeline by Krysztof Dzialo and Leon Haupt (Awais, et al., 2017)

An attempt to draw the existing network in scale with georeferences was made on QGIS to analyze the network's functionalities. Some points have been confirmed and identified on the map from interviews with a local, Jacob Chikuhwa. A few tanks, such as the Samaringa tanks and the Honde clinic tank, have had coordinates set in the 2017 survey. The elevation points found in the digitalized picture have matched the interpreted map's elevation (see Figure 24).

The main pipes of line 2, line 4, and the old line have been assumed to follow certain infrastructures such as roads. Line 3 has been assumed to follow an elevation line around Hwahwazira mountain. Most of the outlets have been assumed to lead to clusters of households found from the satellite view.

The village borders had to be defined to locate the households and their respective water demand. The borders have been determined from an interview with Jacob Chikuhwa in 2021. To broadly confirm the village borders, each household has been located from the satellite view (Google, 2018), whereafter, the number of households found on the map has roughly matched the number of households per village provided by ZINWA.

Table 19, number of households comparison

Village	Inhabitants (ZINWA, December 2020)	Households (ZINWA, December 2020)	Houses marked from satellite view
Ward 10	3687	1957	2262
Cluster 1 (Marked in blue)	1634	816	913
Matingo	889	446	444
Nyakabinga	322	56	97
Chipunha	163	81	87
Saruwaka-Gowa	260	233	286
Cluster 2 (Marked in red)	686	235	335
Samaringa	159	59	90 (school district included)
Mutsaka	162	57	82
Baradza	174	56	97 (Business center included)
Hambira	191	63	67
Cluster 3 (Marked in yellow)	630	213	390 (public buildings included)
Ngwende	341	113	264 (churches, business centers and other public buildings included)
Mutetwa	289	100	126
Cluster 4 (Marked in green)	683	693	619
Manyonho	311	234	261
Nyabadza	153	287	214
Mutsamba	219	172	144

The lack of additional concrete geo-points on the pipeline system and village borders makes this not an entirely reliable, only indicative, map. Also, the elevation data used is a 30-meter LIDAR map (RCMRD, 2018) which has interpolated the in-between elevation values. To make a more precise system analysis, a more detailed survey needs to be done using on-site measurements regarding the height differences between the nodes and tanks and their exact location.

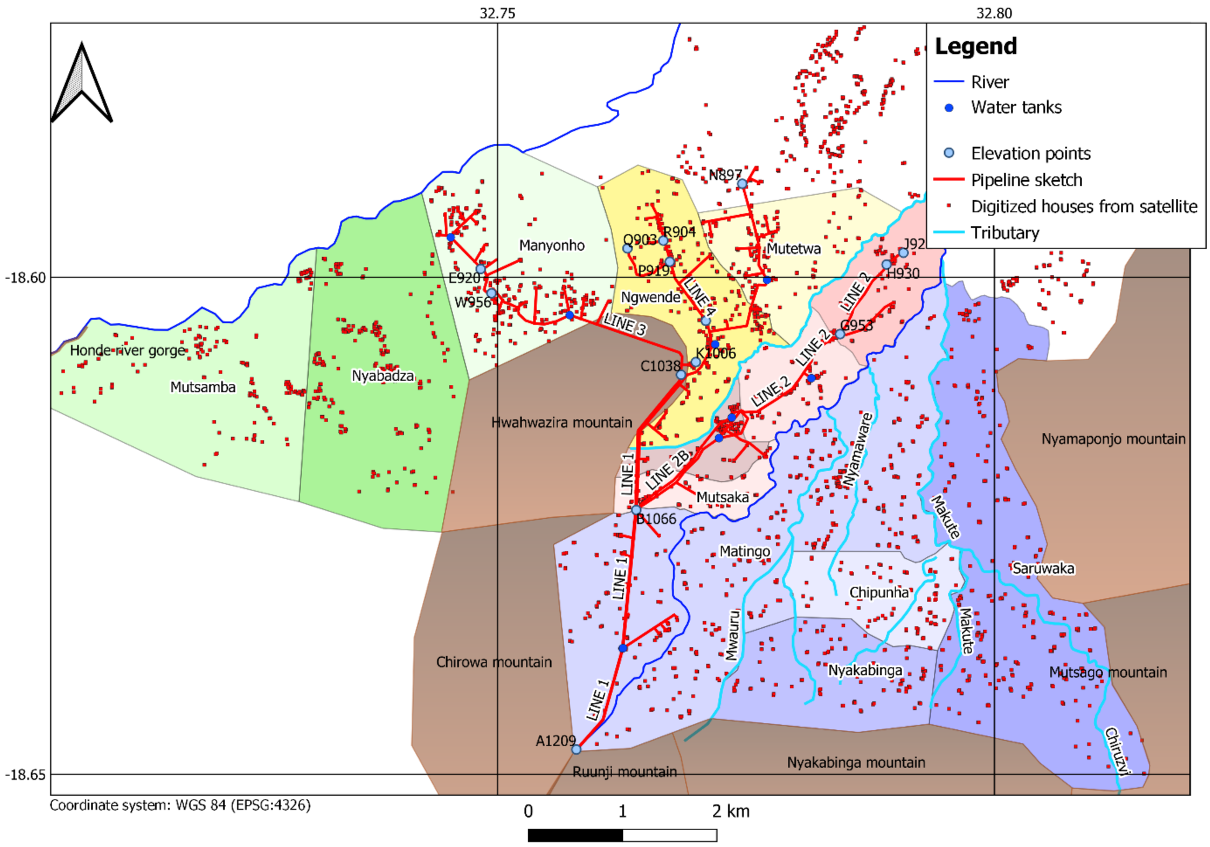

Figure 24. georeferenced interpretation of the existing pipeline network and village borders, map drawn on QGIS

3.3 Water resources in the area

In general, heavy rainfall produces high runoff and good groundwater recharge, especially in mountainous regions with low population densities (Tebbutt, 2002) and, therefore, the rivers and tributaries in Ward 10 could provide a sufficient supply. A rainfall-runoff simulation has been made in QGIS to find the largest rivers and tributaries, which could potentially be used as a water source to the distribution system in Ward 10.

The simulation provides the size of the catchment area, which gives indicative measurements for the capacity of each river and its intake. In essence, the further down the stream an intake is placed, the larger the catchment area is, and the more reliable the river is to provide water to the system. However, the supply capacity must be balanced with the elevation of the intake for it to have sufficient pressure to the gravitationally fed distribution network.

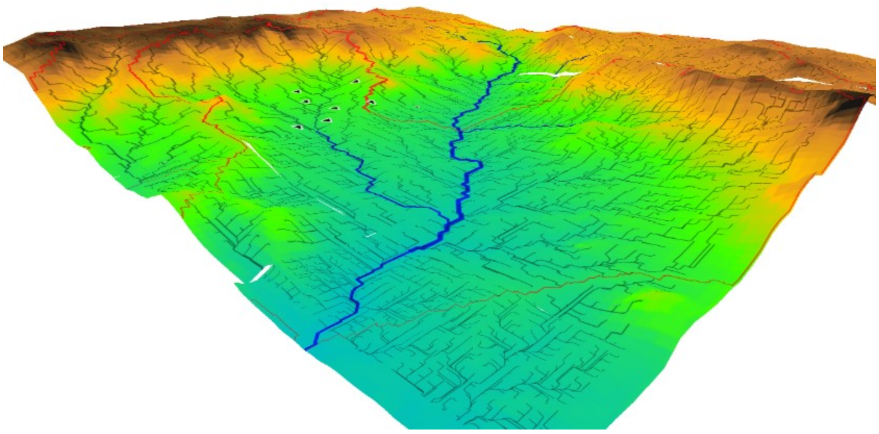

Figure 25, 3D-view of the rainfall catchment areas in Ward 10, QGIS simulation

The simulation only considers the topology of the area and calculates the catchment area and runoff to each tributary; therefore, it is only indicative since the permeability of the soil and soil type has not been taken into account in the simulation. Also, only a resolution of 30 meters elevation LIDAR map is used, which has interpolated the elevation values between the registered elevation points. The simulation results show the location of the tributaries, which has thereafter been confirmed by matching with the satellite pictures and identified through interviews with Chikuhwa.

Figure 26 depicts a detailed map of the rivers and tributaries' catchment areas. The red line divides the catchment area of Honde and Mupenga rivers, whereas the respective smaller tributaries consequently flow to these two larger rivers. The more black arrows that are pointed to a river, the stronger the rainfall-runoff is for the respective river. As can be noted, the larger the area these black arrows cover, the larger the flow of the river is.

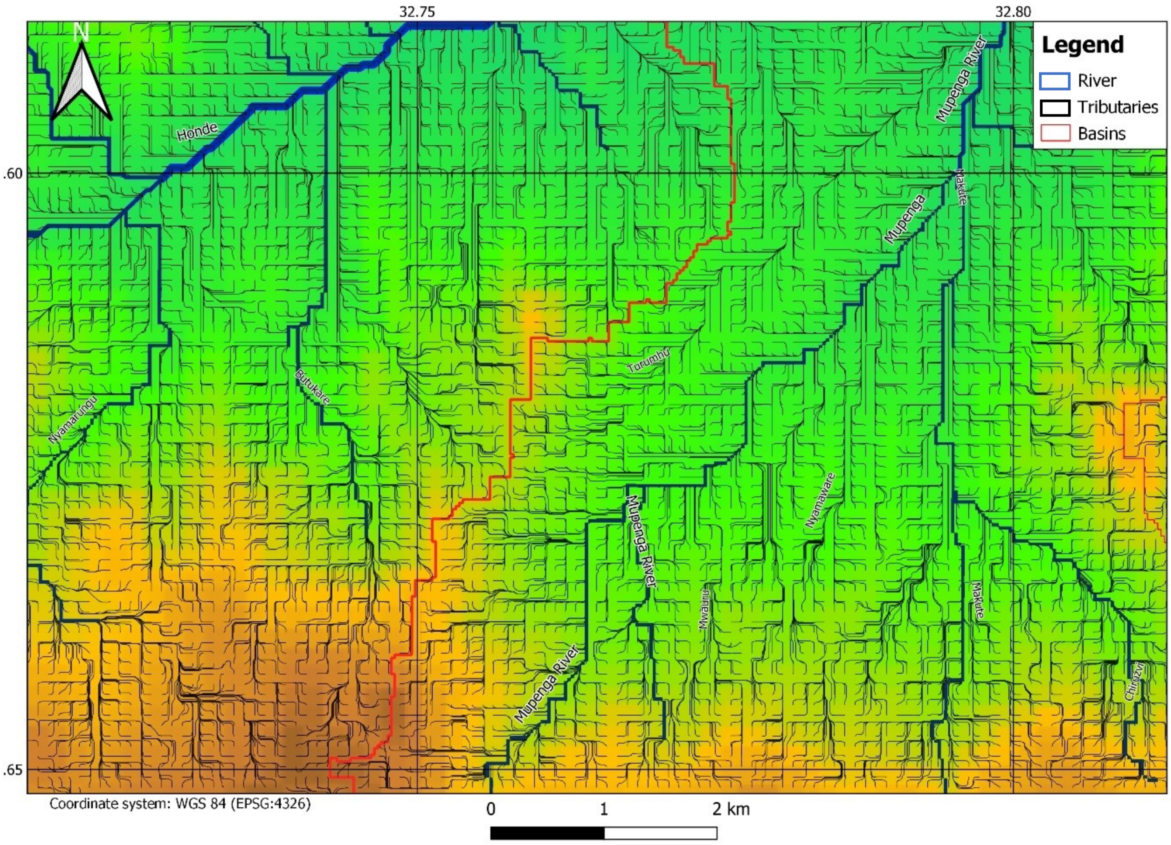

Figure 26, detailed representation of the rainfall-runoff simulation on QGIS of Ward 10

The largest tributaries and rivers identified from the simulation have been measured with a simple on-site method to find their average flow. By measuring the width (w) and depth (d) of the river and timing (t) how long it takes for a floating object to travel a certain distance (l), the results of the flow ($\dot{V}$) for each river have been acquired, as seen in Table 20. Each measurement has been done at three different locations of the respective river, whereafter an average result of the three measurements has been calculated. The measurements were done during the dry season in June 2021 by Jacob Chikuhwa.

$$\dot{V} = \frac{w \cdot d \cdot l}{t} \tag{15}$$

Table 20, Rivers and tributaries flow data

River	Measured flow in June 2021, (l/s)	Drainage basin area km^2(simulation)
Mupenga River	0.91	52
Turumhu Tributary	0.63	3.7
Honde River	1.28	313
Butukare Tributary	0.49	9
Nyamarungu Tributary	>0.49	11
Chiruzvi Tributary	0.62	6
Mwauru Tributary	0.69	4.2
Nyamaware Tributary	0.59	13.5

Groundwater sources are usually preferred as intakes over surface water sources since the water quality is generally better in groundwater sources, especially regarding bacteriological contamination. A groundwater simulation has been done in the area to locate potential boreholes and spring catchments where the recharge rate is sufficient. However, the data for this simulation have been limited, and further analyzes are required to confirm the actual potential use of these wells. In Figure 27 and Table 21, one to three boreholes per village cluster have been located from the simulation, which could potentially be used as additional intakes to the distribution system (Thomas, 2021).

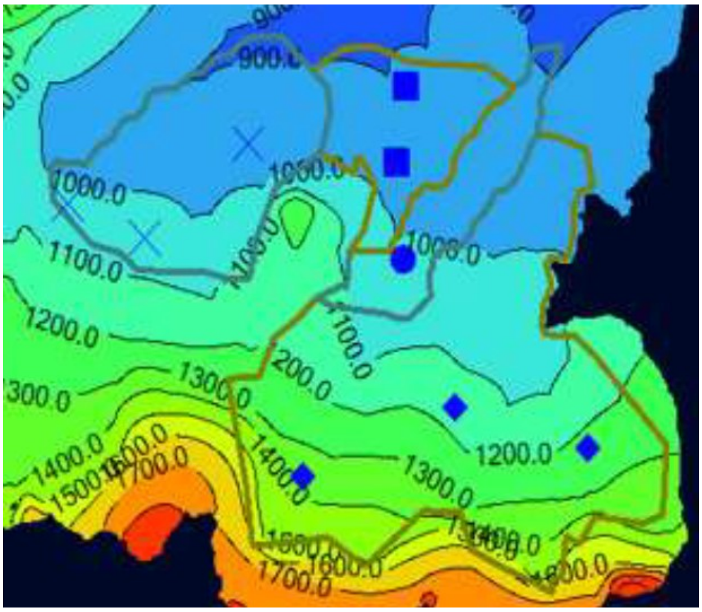

Figure 27, potential wells in Ward 10 (Thomas, 2021)

Table 21, well recharge rate (Thomas, 2021)

Well ID	Recharge rate (m^3/day)
Cluster 1, three wells	51 516
Cluster 2, Samaringa borehole	8414
Cluster 3, two wells, (whereas the southern is Patsime water hole)	10 353
Cluster 4, four wells	19 887

Locals already use two of the identified potential wells in the groundwater simulation: the Patsime water hole in village cluster 2, which is an artesian spring (see Figure 28) and the Samaringa borehole, which provides large quantities of water to the Samaringa village.

Figure 28, locals fetching water from Patsime water hole (Chikuhwa, 2021)

4 Water supply proposition to Ward 10

The locals do not rely on the current public pipeline. Therefore, they have built their own disorganized haphazardous systems to supply their respective needs for irrigation to support their livelihood and drinking. The proposition is to build a new reliable pipeline system to meet the water demand throughout Ward 10 with a public gravitationally fed distribution regulated and maintained under trained personnel.

The three fundamental technical flaws of the old networks are, firstly, that the Mupenga River's supply (0.91 l/s) is not sufficient to cover the area that the pipeline network covers (average base demand 1.27 l/s) and, secondly, that the pipeline does not cover all of Ward 10, only some villages. Therefore, additional intakes to the existing network are needed, and an expansion of the pipeline system with more intakes to the remaining villages is needed. Thirdly, the old pipeline system and the self-built haphazardous systems do not have any functioning treatment and distribute raw water to the outlets, leaving the treatment responsibility to the user.

The layout of the old network is fairly well planned out, as can be seen in Figure 31, where its outlets cover most of the inhabited area it is designed to cover. However, due to its unknown condition, it may need to be rebuilt or at least repaired. Therefore, the new proposition has only two proposed changes to the existing pipeline system: Firstly, an additional intake from Patsime water hole supplies water to Ngwende and Mutetwa with a pump to a storage tank on the nearby hill, whereafter it is gravitationally distributed to cluster 3. Secondly, to ensure sufficient water supply to Cluster 2, the Samaringa borehole is proposed to be connected to the system via line 2B. Therefore, the water supply from Mupenga River is sufficient when distributing water only to Manyonho village via line 3 and cluster 2 via line 2 with the complementary borehole intake at Samaringa. Line 5 to 9 are the new proposed distribution lines covering the rest of village clusters 1 and cluster 4. Due to the area's topography, the proposed networks are not connected to each other but separate systems. Line 5 to line 7 supply most of village cluster 1, and line 8 and line 9 supply water to Nyabadza and Mutsamba. A more detailed description of each line is presented in Table 22, and the lines are depicted in Figure 29.

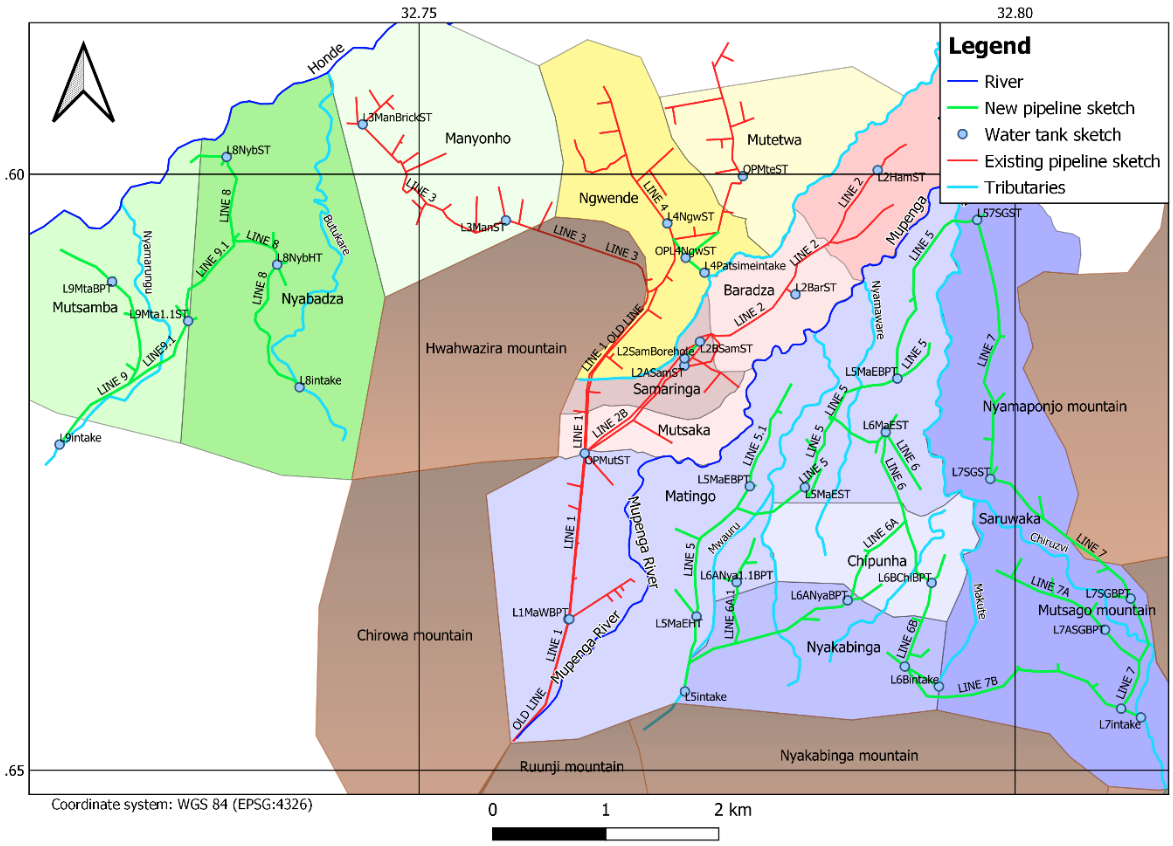

Figure 29, proposed pipeline layout overview, red lines are the old system and green lines are the new proposed pipelines

Table 22, detailed description of each line

LINE	Pipe layout description
Old Pipeline in Matingo	The OP supplies Line 2 with water from Mupenga river via the large storage tank. Along its way are some outlets in the west part of Matingo.
Old Pipeline in Ngwende and Mutetwa	The OP supplies mainly Mutetwa village and Line 4 with water from Turumhu river, alternatively with pumping power from Patsime water hole to a new storage tank on the nearby hill.
LINE 1	Line 1 runs in parallel with OP line in Matingo and supplies water to Line 3 from Mupenga river.
LINE 2A/B	Line 2 A and B follows a road from the large storage tank and supplies water to Mutsaka and the Samaringa school district,

	whereafter it connects with Line 2. Line 2B is supplied with water from the Samaringa borehole to complement the supply.
LINE 2	Line 2 receives water from Line 2A and B and continues along the road through Baradza and Hambira where it supplies water.
LINE 3	Line 3 receives water from Line 1 and follows an elevation curve around Hwahwazira mountain from where it distributes water to Manyonho village.
LINE 4	Line 4 receives water from either Turumhu river or Patsime waterhole and distributes water to Ngwende village.
LINE 5	Line 5 Supplies water to Matingo village from Mwauru river. The line follows a road which is in parallel with the river, wherafter it crosses the Mwauru river and supplies water to a storage tank near the center of Matingo village. The line continues along a road and crosses Nyamaware river and Makute river until its end point in the Saruwaka Gowa storage tank.
LINE 5.4	Branch line 5.4 continues down the parallel road with Mwauru and supplies water to the inhabitants between Mwauru and Mupenga river.
LINE 6A	Line 6A shares its intake supply with Line 5 from Mwauru river. It crosses two rivers and distributes water to the western part of Nyakabinga and Chipunha. At the villages borders it follows a road that leads to Line 6.
LINE 6A1	Line 6A1 is a branch from Line 6A which distributes water to the households east of Mwauru.
LINE 6B	Line 6B receives water from Makute river, however if the supply is insufficient it can have an additional supply from Line 7B. The line distributes water to the eastern part of Nyakabinga and Chipunha following a road until it connects to Line 6.
LINE 6	Line 6 combines Line 6A and Line 6B. It follows a road and provides water to a storage tank on a hill which subsequently connects to line 5 and supplies water to Matingo.
LINE 7	Line 7 supplies water to Saruwaka- Gowa village from Chiruzvi river. It follows an elevation curve from Mutsago mountain and

	continues along Nyamaponjo mountain to the end of the Saruwaka Gowa village.
LINE 7A	Line 7A follows the road beside Chiruzvi river in order to make construction easier. Line 7A supplies water to the inhabitants west of Chiruzvi in Saruwaka-Gowa.
LINE 7B	Line 7B follows an elevation curve from the Chiruzvi intake to supply additional water to Line 6B. Along its way are some outlets to the nearby inhabitants in Saruwaka Gowa.
LINE 8	Line 8 supplies water to Nyabadza from Butukare river. The storage tank in the village center may need pumping power to fill since Butukare river is at such a low elevation. The line follows the road throughout the village.
LINE 9	Line 9 provides water to Mutsamba from Nyamarungu river. The line follows the road west om Nyamarungu at a set elevation.
LINE 9.1	Line 9.1 is the link between Mutsamba village and Nyabadza and supplies additional water to the intermediate households before it connects to Line 8 via a storage tank.

A simplified overview of the respective lines' components is illustrated in Figure 30. The demand of the outlets and supply of the intakes for the respective line decide the dimensions and design flows for their respective components.

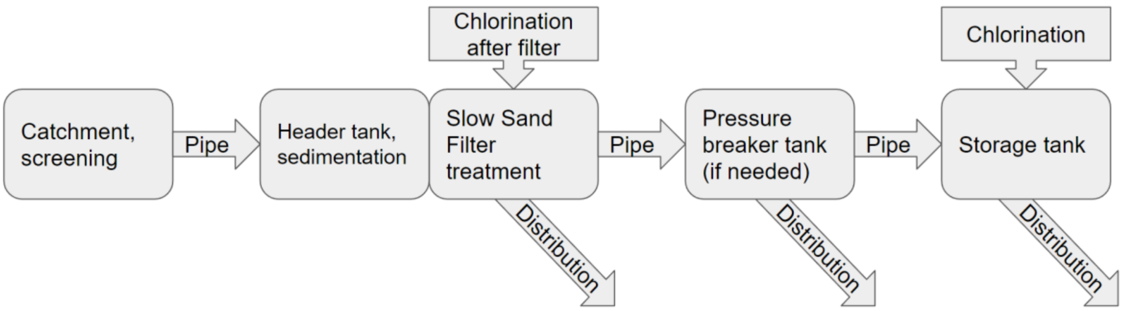

Figure 30, line component overview

4.1 Demand specifications

The overview map in Figure 31 shows the proposed outlets positions of the whole pipeline system, where the outlets are located in the middle of each green circle. The green circles have a radius of 250 meters which is the maximum distance between a household and an outlet. The pipeline system's design aims to cover as large of an area as possible without breaking hydraulic rules, which is also how the number of outlets has been determined in Table 23.

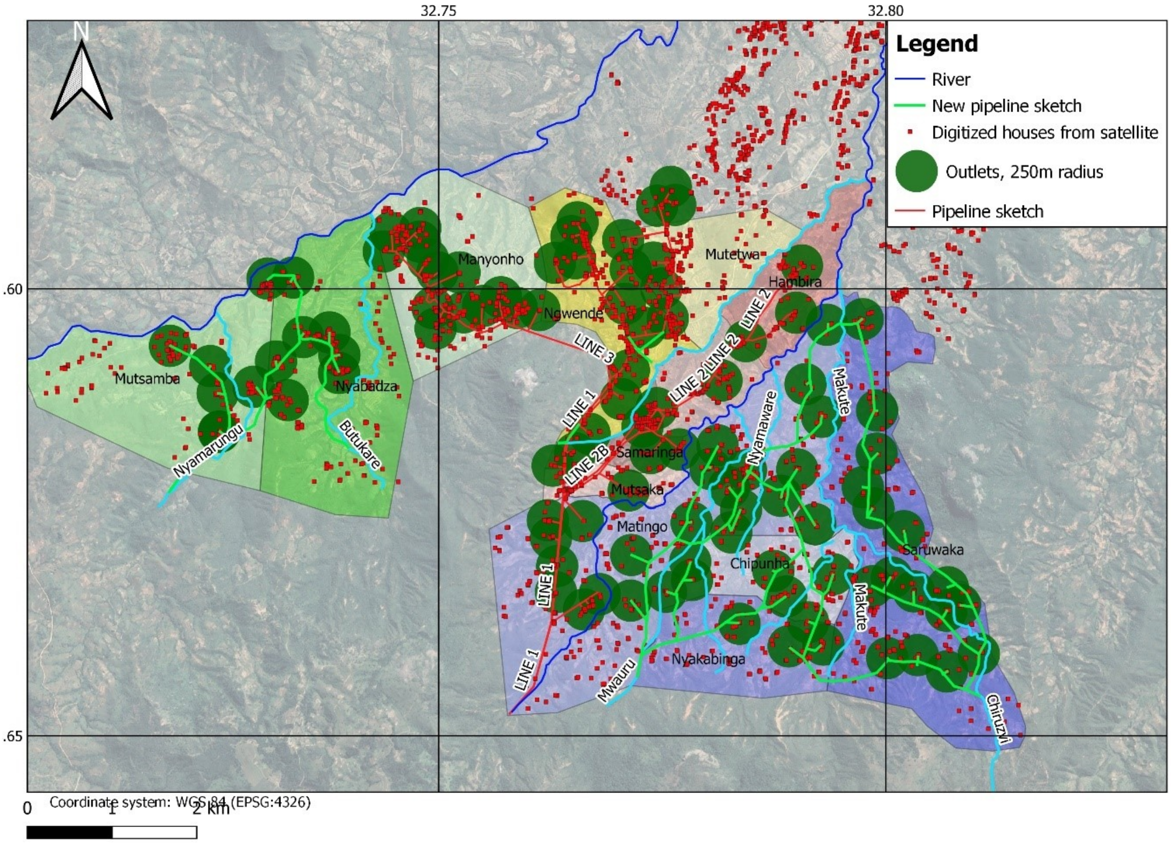

Figure 31, Pipelines distribution area and outlets locations

The base demand is estimated from a 30-year perspective with a 1.48% population growth (The World Bank, 2021) for 80 liters of water per inhabitant per day. The worst-case scenario multiplies the base demand with a general factor of 4.2 to compensate for seasonal patterns and other unpredicted changes in the demand.

Table 23, water demands

Cluster	Village	Inhabitants by ZINWA (2017)	Predicted number of inhabitants in 30 years	Base demand per village (80 liters per capita per day)	Number of outlets	Base demand per outlet (l/s)	Demand per outlet, with multiplier 4.2 (l/s)
1	Matingo west	195	303	1.28	9	0.02	0.084
	Matingo east	690	1072	1.28	15	0.04	0.168
	Chipunha	163	253	0.23	4	0.04	0.168
	Nyakabinga	322	500	0.46	3	0.15	0.63
	Saruwaka Gowa	260	404	0.37	14	0.027	0.1134
2	Mutsaka	162	252	0.23	2	0.117	0.4914
	Samaringa	159	247	0.23	5	0.046	0.1932
	Baradza	174	270	0.25	2	0.13	0.546
	Hambira	191	297	0.27	2	0.14	0.588
3	Ngwende	341	530	0.49	12	0.04	0.168
	Mutetwa	289	449	0.42	11	0.038	0.1596
4	Manyonho	311	483	0.45	16	0.028	0.1176
	Nyabadza	153	237	0.22	6	0.037	0.1554
	Mutsamba	219	340	0.32	5	0.063	0.2646

ZINWA provided the water irrigation storage demand for Ward 10, which is in total 471600 m^3 storage required per irrigation cycle, when 50 irrigation cycles for a period of six months average one irrigation cycle every 72h. Therefore, the average water supply needed for irrigation purposes to Ward 10 is 1.82 l/s. When divided by the predicted inhabitants, 5645, equals 28 liters per capita per day, whereas the remaining 52 liters per capita per day is for domestic use. Therefore, the 80 liters per capita per day demand is sufficient and applies the WHO standards according to Figure 3 where 50 liters per capita per day of potable water is provided for domestic use and 30 liters per capita per day for irrigation.

No deeper analysis has been done regarding the demand and supply patterns in Ward 10. Therefore, the generic theoretical patterns, presented in section 2.2.1, are used as demand patterns, and constant values from the dry season are used for the supply pattern. This assumption may affect the dimensions of the system, especially the water tanks in section 4.4.

4.2 Catchments

Spring catchments are typically used for gravity-fed systems due to their consistent flow and good water quality. However, additional surveys of the groundwater flow need to be done to find more possible groundwater sources and spring catchments in Ward 10. Therefore, due to the provided data, surface water sources have been used as intakes to the proposed system; thus, water treatment is recommended.

The proposed potential intakes to the system have had their supply capacity data measured during the dry season (see Table 24). Since the supply must be larger than the demand and the river flow data are measured once during the dry season when the flow is at its yearly minimum, multiple intakes are proposed to meet the supply and demand criteria. Fewer intakes could be used, and water could be stored throughout the dry season in larger storage tanks if more thorough surveys on the rivers' seasonal variations are done. Another restricting criterion to the number of intakes to the system is the area's topography. It is sought to provide the water gravitationally to the network, limiting the layout of the network and placement of the intakes, except for two wells, Patsime water hole, and Samaringa borehole, which require pumping power to be connected to the network.

The exact location (see Figure 29) and capacity of each proposed intake needs to be further analyzed. If the supply from a river is insufficient at the proposed location, it can potentially be sufficient further down the river as the rainfalls catchment area increases further down the river, as seen in the simulation in section 3.3. However, the intake's supply and elevation need to be balanced for the gravity-fed network to function.

It is recommended to build the intakes from the rivers with indirect catchments, which would provide a better and safer quality of water to the distribution system, as stated in section 2.4.1.

Table 24, water supply and demand for the network

<table>
<tr><th colspan="2">Source / supply (avg. l/s)</th><th>Distribution / total demand (avg. l/s)</th></tr>
<tr><td colspan="2">Mupenga river / 0.91
Samaringa borehole / 97 (recharge rate)</td><td>Cluster 2, Manyonho, Matingo west / 1.62</td></tr>
<tr><td colspan="2">Patsime water hole / 120 (recharge rate)</td><td>Cluster 3 / 0.9</td></tr>
<tr><td>Chiruzvi / 0.62</td><td rowspan="3">Total supply est. 2</td><td rowspan="3">Cluster 1 (Matingo west not included) / 2.17</td></tr>
<tr><td>Makute / no data</td></tr>
<tr><td>Mwauru / 0.69</td></tr>
<tr><td colspan="2">Butukare / 0.49</td><td>Nyabadza / 0.22</td></tr>
<tr><td colspan="2">Nyamarungu / >0.49</td><td>Mutsamba / 0.32</td></tr>
</table>

Mupenga River supplies water to Cluster 2, Manyonho, and Matingo west. Due to the insufficient supply, cluster 2 can use the Samaringa borehole as an additional intake to line 2 via tank L2BSamST. The remaining needed supply has a base demand of 0.394 l/s, with a worst-case scenario multiplier 4.2 which gives 1.65 l/s. The estimated depth of the borehole is 30 meters. Required pumping power for the base demand is 0.12 kW, respectively 0.5 kW for a worst-case scenario demand. It is calculated with equation 11, from section 2.4.1.

Cluster 3 is mainly supplied from the Patsime water hole, which has a base demand of 0.74 l/s; worst-case scenario 3.1 l/s. The water needs to be pumped from the water well to the storage tank (OPL4NgwST), which has an elevation difference of 23 meters from the well to the hill, added with the storage tank's water levels, gives an estimated elevation difference of 30 meters. To meet the required base demand 0.2 kW pumping power is needed, whereas the maximum pumping power required is 0.9 kW.

Pump curves and pipe curves should be estimated for the pumps' dynamic regulation and steering.

4.3 Pipe specifications

The topographical profile and pressure profiles of the line sections between the tanks of each line are illustrated in the profile plots in Appendix B. Their locations are depicted in Figure 29.

The design flow is the sum of the worst-case scenario demand for every outlet the respective pipe covers.

Maximum pressure is the highest static pressure found in the pipeline (see 9.2 Appendix B, Pressure profiles and pressures in pipelines), whereafter a suitable standardized NP pipe is chosen. However, the pressure values in the simulation assume the tank's water level to equal the surface level, which in practice means that the tanks are empty, which therefore assumes a lower pressure in the system than the actual.

The pipe lengths have been determined from the QGIS map.

The pipe diameter is chosen by the design flow and the elevation difference between the end points for the respective line. The Hazen-Williams head losses have been subtracted from the elevation difference, whereafter a suitable diameter has been chosen, but not smaller than 63mm. The diameters have then been implemented to the simulation in MIKE+ to confirm the functionality of the hydraulic behavior in the system.

Table 25, pipe data

Lines and respective sections	Design flow per section (l/s)	Maximum Pressure in line (m)	Required Pipe PN	Pipe length (m)	Pipe inner diameter (mm)
Old Pipeline, Mupenga to Ngwende					
- Mupenga intake to PBT	4	85	100	1262	110
- PBT to OPMutST	4	55	64	1526	90
- OPMutST to end	1.6	60	64	1507	90
- Branches	-	-	64	2157	63
Old Pipeline, Patsime to Mutetwa					
- Patsime intake to OPMteST	1.6	60	64	1207	63
- OPMteST to end	1.6	60	64	1304	63
- Branches	-	-	64	2271	63
LINE 1					

- Mupenga intake to L1MaWPBT - L1MaWPBT to L3ManST - Branches	2 2 -	80 170 -	100 250 250	1255 4760 600	90 75 63
LINE 2A+B - From OPMutST to Line 2 - Branches	4 -	70 -	100 100	1556+1611 1591	75 63
LINE 2 - From Line 2AB to end - Branches	2.5	80	100	2610 786	63 63
LINE 3 - L3ManST to end - Branches	4 -	70 -	100 100	2271 2488	63 63
LINE 4 - Patsime intake to end - Branches	1 -	90 -	100 100	1965 2033	63 63
LINE 5 - From intake to HT - From HT to ST - From ST to PBT - From PBT to ST - Branches	3.5 3.5 1.3 0.6 -	60 85 85 55 -	64 100 100 64 100	706 1857 1440 1889 1693	90 90 75 63 63
LINE 5.4 - From line 5 to end - Branches	0.5 -	100 -	100 100	1261 85	63 63
LINE 6A - From intake to line 6 - Branches	2 -	55 -	64 64	2486 549	90 63
LINE 6A1 - From line 6A to end - Branches	0.5 -	90 -	100 100	1031 205	63 63
LINE 6B - From intake to line 6 - Branches	0.7 -	90 -	100 100	1479 751	63 63
LINE 6	1	95	100	867	75

- From line 6AB to ST - From ST to Line 5 - Branches	1 -	75 -	100 100	576 888	75 63
LINE 7 - From intake to PBT - From PBT to ST - From ST to ST - Branches	1 0.6 0.6 -	155 70 140 -	160 100 160 160	1240 1756 2470 856	90 75 63 63
LINE 7A - From line 7 to PBT - From PBT to end - Branches	0.5 0.5 -	130 45 -	160 64 -	380 1163 456	63 63 63
LINE 7B - From HT to line 6B - Branches	2.6 -	35 -	40 40	2267 52	63 63
LINE 8 - Intake to HT - HT to end - Branches	0.7 0.5 -	35 80 -	40 100 100	1373 1663 568	63 63 63
LINE 9 - From intake to PBT - From PBT to end - Branches	1 1 -	75 30 -	100 40 100	516 1922 226	63 63 63
LINE 9.1 - From line 9 to ST - From ST to line 8 - Branches	1 0.5 -	60 30 -	64 40 64	818 932 327	63 63 63

Total needed pipe length for the distribution system is 71.5 km, Table 26 summarizes the different pipe specifications required. An estimated average price of $80 per 100 meters of HDPE pipe equals a total pipe cost of $57 200.

Table 26, total pipe material

	Ø 63mm	Ø 75mm	Ø 90mm	Ø 110mm
PN40	6546m	-	-	-
PN64	11685m	-	6225m	-
PN100	24110m	7806m	3112m	1262m
PN160	4162m	-	1240m	-
PN250	600m	4760m	-	-

4.4 Water tank specifications

The respective water tank's location is depicted in Figure 29.

The header tanks' sizes are dimensioned according to the sedimentation tanks calculation theories in section 2.3.3.1. Fine silt particles ($d = 16$ μm, $V_s = 0.0018$ m/s) are assumed to settle with an HRT of 28 minutes.

The storage tanks' dimensions are based on the sum of the maximum inflow and maximum outflow of the largest variation for the selected time frame. The pattern used for the minimum daily domestic variation is the same as in Figure 4. The pattern used in the theoretical variation with two irrigation cycles is the same as Figure 5, which is a time frame of six days.

BPT: s are all two cubic meters according to the theory in section 2.4.3.

Most of the tanks are quite small, hence could the local plastic tanks be suitable for some as the ones in Figure 32. Larger tanks (>15 m^3), BPT: s, and HT: s can be constructed with concrete or bricks, which would require further construction costs; however, the rough cost estimate for the larger tanks has interpolated the price value as to be similar to the plastic tanks. The costs are based on the sizes from Table 28.

Table 27, locally available plastic tanks price tags from LAMASAT (Awais, et al., 2017)

Cost ($)	Size (m^3)
260	2
440	5
720	7.5
1100	10

Table 28, required water tank sizes

Tank ID	Existing tank volume (m^3)	Minimum daily domestic use volume variation (avg. demand) (m^3)	Theoretical variation of two irrigation cycles (m^3)	Material cost ($)
OPMaWBPT (calculated as a HT)	16.3	7	7	720
OPMutST	372.6	13	44	4400
OPL4NgwST	Derelict	12	35	4400
OPMteST	340	6	18	2200
L1MaWBPT (Calculated as a HT)	16.3	3	3	260
L2ASamST	168.65	12	37	4400
L2BSamST	168.65	6	18	2200
L2BarST	33.2	2	6	720
L2HamST	39.3	2	6	720
L3ManST	227.5	6	18	2200
L3ManBrickST	227.5	2	7	720
L4NgwST	153	3.5	11	1100
L5MaEHT	-	3.5	3.5	260
L5MaEBPT4	-	2	-	260
L5MaEST	-	3	10	1100
L5MaEBPT	-	2	-	260
L57SGST	-	2	2	260
L6ANya1.1BPT	-	2	-	260
L6ANyaBPT	-	2	-	260
L6BNyaHT	-	2	2	260
L6BChiBPT	-	2	-	260
L6MaEST	-	3	10	1100
L7SGHT	-	4.3	4.3	440
L7SGBPT	-	2	-	260
L7SGST	-	2	6	720
L7ASGBPT	-	2	-	260
L8NybHT	-	2	2	260
L8NybST	51.89 (location unknown)	2	4	440
L9MtaHT	250 (location unknown)	2	2	260
L9MtaBPT	-	2	-	260
L9MtaST1.1	-	3	11	1100

The obtained values in Table 28 of the water tanks' variation of volumes must still be analyzed, since the demand patterns used are theoretical, and the weekly and seasonal variations have not been taken into account due to lack of data. In addition to the theoretical demand pattern, a seasonal supply pattern is needed to determine the need for storage throughout the seasons. The water supply used in this calculation is constant from the dry season (see Table 20).
For the short-term variation demands, suitable sized tanks for the potable water system can be found locally, as shown in Figure 32.

Figure 32, Water tanks from a local producer (Awais, et al., 2017)

4.5 Water treatment specifications

The distribution network is dimensioned for 80 liters per capita per day, which combines domestic water use and irrigation demand into one network. Therefore, whether a centralized treatment for both water qualities with the determined design flow is economically beneficial needs to be surveyed. Alternatively, two parallel systems can be built for the respective water quality, but it doubles the material cost.

However, water treatment is recommended for domestic water use due to the intakes being mainly from surface water. The proposed treatment setup is highly dependent on the raw water quality analysis; due to the lack of water quality analyses from the area, the proposed water treatment included all steps necessary in case of bad water source quality. The treatment methods may also vary between the intakes due to different local water qualities.

4.5.1 Analysis

Before selecting the intakes' locations, the quality should be surveyed. Field method analyses such as EC, turbidity, pH, and biological contaminations can easily be tested for each intake if the equipment is procured, whereafter ZIMLABS in Harare have stock analyses for 60$ per sample, which test the presence of chemicals (Ca, Mg, Na, K, Mn, Cu, Zn, Fe, Cl, SO4, NO_3), pH, EC, TDS, TH, Ecoli and TVC. Additionally, samples should be taken during different seasons, especially regarding the turbidity of the water, which can vary strongly between seasons.

Continuous analysis is needed for the distribution system to ensure good quality and measure chlorine residuals to find suitable amounts of disinfections needed for the system.

4.5.2 Primary step

The catchment from the rivers can be constructed in different ways, the easiest of which is a dam with an inlet with a physical filter with 25mm holes in it. A preferable method is an indirect collection of the river water, as illustrated in Figure 13. This method would not necessarily require a sedimentation tank and possibly not a sand filter either, depending on the quality.

From videos and pictures, the water sources in Ward 10 seem to have low turbidity; however, it is not taken for granted. If the turbidity of the raw water is >10 NTU, then pre-treatment is recommended before the secondary filtration step not to clog the sand filters. A sedimentation basin is relatively cheap and straightforward to build. In the proposed system, the header tanks are designed to work as sedimentation tanks, since the header tanks are always the first tanks before any outlet in the system. The dimensions calculated in Table 29 have an HRT of 28 minutes for fine silt particles with a 0.0018 mm diameter to settle.

Table 29, sedimentation tank dimensions

Intake and header tank	Design flow (l/s)	Header tank size and dimensions h (m) x w (m) x l (m) = V (m^3)
Mupenga river - OPMaWBPT - L1MaWBPT	 4 2	 3 x 0.75 x 3 = 7 3 x 0.5 x 2 = 3
Mwauru - L5MaEHT	 3.5	 3 x 0.7 x 3 = 6.3
Makute - L6BNyaHT	 0.7, used 1 l/s due to small dimensions	 3 x 0.4 x 1.6 = 2
Chiruzvi - L7SGHT	 2.6	 3 x 0.6 x 2.4 = 4.3
Butukare - L8NybHT	 0.7, used 1 l/s due to small dimensions	 3 x 0.4 x 1.6 = 2
Nyamarungu - L9intake	 1	 3 x 0.4 x 1.6 = 2

4.5.3 Secondary step

A slow sand filter can be combined with the header tank in the system to effectively treat the water before entering the distribution system. A slow sand filter can work as a single treatment step without pre sedimentation if the raw quality water is good, i.e.,>10 NTU and low concentration of chemicals. The dimensions for slow sand filters depend on the design flow for the respective line divided with 0.3 m^3/h per square meter. The total needed filter size for the maximum design flow is shown in Table 30; however, the required size can be divided into several smaller parallel filters, enabling easier maintenance while keeping a continuous flow in the system.

The construction costs for SSF: s can be estimated to \$100-\$300/m^2 (sswm, 2021).

Table 30, SSF dimensions

Intake and header tank	Design flow (LPS)	Filter size (m^2)	Cost (\$), \$100/m^2
Mupenga river			
- OPMaWBPT	4	48	4800
- L1MaWBPT	2	24	2400
Patsime water hole*			
- OPL4NGWST	1.6	19.2	1920
Samaringa borehole*			
- L2BSamST	1	12	1200
Mwauru			
- L5MaEHT	3.5	42	4200
Makute			
- L6BNyaHT	0.7	8.4	840
Chiruzvi			
- L7SGHT	2.6	31.2	3120
Butukare			
- L8NybHT	0.7	8.4	840
Nyamarungu			
- L9intake	1	12	1200

The total cost for installing SSFs to the required square meters is \$20,520.

The Samaringa borehole and Patsime waterhole intakes may not necessarily need an SSF due to their natural groundwater quality. The necessity depends on the water quality analyses, although the EC analyses conducted in 2017 from the Samaringa borehole and Patsime water hole have a relatively high conductivity which might indicate high turbidity.

4.5.4 Tertiary step

Chlorination is largely dependent on the water quality in the system. Without further water analyses, an estimation of the amount needed for respective tank can be calculated from Table 31. The chlorine residuals should be between 0.1 and 0.3 mg/l in the system, unless shock chlorination is needed, which leaves 50 mg/l chlorine residuals. The chlorine should sit in the system for 12-24 hours. Two different concentrations of disinfection are reviewed, liquid bleach and powdered calcium hypochlorite (Clean Water Systems, 2017).

Table 31, chlorination demand approximation (Clean Water Systems, 2017)

Tank size (m^3)	Shock chlorination, chlorine residual concentration 50 mg/l		Routine chlorination, chlorine residual concentration 1 mg/l	
	12.5% bleach sodium hypochlorite (liters)	65% powdered calcium hypochlorite (liters)	12.5% bleach sodium hypochlorite (liters)	65% powdered calcium hypochlorite (liters)
1.9	1	0.2	0.04	0.01
3.8	2	0.35	0.07	0.02
9.5	4	0.7	0.2	0.04
19	8	1.5	0.4	0.1
38	15	3	0.7	0.2

The additional chlorine residual concentration 1 mg/l can compensate for the pipelines to make the final chlorine residual concentration 0.1-0.3 mg/l. Header tanks should not be chlorinated if they are combined with SSFs, however, in Table 32 they are included to assume the full water volume in the system to approximate the amount of disinfectant needed.

Table 32, chlorination demand for the proposed system

Tank ID	Theoretical tank size (m^3)	Shock chlorination, chlorine residual concentration 50 mg/l		Routine chlorination, chlorine residual concentration 1 mg/l	
		12.5% bleach sodium hypochlorite (liters)	65% powdered calcium hypochlorite (liters)	12.5% bleach sodium hypochlorite (liters)	65% powdered calcium hypochlorite (liters)
OPMaWBPT (calculated as a HT)	7	3	0.5	0.13	0.03
OPMutST	44	15	3	0.7	0.2
OPL4NgwST	35	15	3	0.7	0.2
OPMteST	18	8	1.5	0.4	0.1
L1MaWBPT	3	2	0.35	0.07	0.01
L2ASamST	37	15	3	0.7	0.2
L2BSamST	18	8	1.5	0.4	0.1
L2BarST	6	3	0.5	0.13	0.03
L2HamST	6	3	0.5	0.13	0.03
L3ManST	18	8	1.5	0.4	0.1
L3ManBrickST	7	3	0.5	0.13	0.03
L4NgwST	11	4	0.7	0.2	0.04
L5MaEHT	3.5	2	0.35	0.07	0.02
L5MaEBPT4	2	1	0.2	0.04	0.01
L5MaEST	10	4	0.7	0.2	0.04
L5MaEBPT	2	1	0.2	0.04	0.01
L57SGST	2	1	0.2	0.04	0.01
L6ANya1.1BPT	2	1	0.2	0.04	0.01
L6ANyaBPT	2	1	0.2	0.04	0.01
L6BNyaHT	2	1	0.2	0.04	0.01
L6BChiBPT	2	1	0.2	0.04	0.01
L6MaEST	10	4	0.7	0.2	0.04
L7SGHT	4.3	2	0.35	0.07	0.02
L7SGBPT	2	1	0.2	0.04	0.01
L7SGST	6	3	0.5	0.13	0.03
L7ASGBPT	2	1	0.2	0.04	0.01
L8NybHT	2	1	0.2	0.04	0.01
L8NybST	4	2	0.35	0.07	0.02
L9MtaHT	2	1	0.2	0.04	0.01
L9MtaBPT	2	1	0.2	0.04	0.01
L9MtaST1.1	11	4	0.7	0.2	0.04
Total	**283 m^3**	**120 liters**	**22.6 liters**	**5.5 liters**	**1.4 liters**

4.6 Financial aspect

The financial analyses present the concrete material costs for the proposed system and its estimated construction costs. Due to the proposal having several separate networks in different villages, these can be evaluated separately if other means of water supply are more suitable for the respective villages. Furthermore, it is assumed that the old network needs complete refurbishment. Hence, all the necessary equipment for the old system is included in the cost calculation. The prices for the respective components have been reviewed in chapter 4, except the valves and accessories which is an estimate.

Table 33, material costs of pipelined system

Component	Old system, OP, L1, L2, L3, L4 ($)	Cluster 1, L5, L6, L7 ($)	Nyabadza, Mutsamba, L8, L9 ($)	Total cost ($)
Pipes	27 808	22 722	6676	57 206
Tanks	24 040	5960	2320	32 320
SSF	10 320	8160	2040	20 520
Valves and accessories	4500	3500	2000	10 000
Total material cost	66 668	40 342	13 036	120 036

The laying of the pipeline, such as excavation of the pipelines, sand bed protection around the pipes and installation of the pipes and valves, can be estimated with the following ratio (Table 34) (Arnalich, 2020):

Table 34, total project estimate cost

Ratio	Old system, OP, L1, L2, L3, L4 ($)	Cluster 1, L5, L6, L7 ($)	Nyabadza, Mutsamba, L8, L9 ($)	Total pipeline costs ($)
Pipes, valves and accessories 36%	32 308	26 222	8676	67 206
Excavation of pipes 31%	27 820.78	22 580.06	7471	57 871.83
Sand bed (pipe protection) 16%	14 359.11	11 654.22	3856	29 869.33
Valve boxes 11%	9871.89	8012.28	2651	20 535.17
Pipe installation 6%	5384.67	4370.33	1446	11 201
Total pipeline layout costs	89 744.44	72 838.89	24 100	186 683.3
Total project cost (Pipeline layout + Tanks + SSFs)	**124 104.4**	**86 958.89**	**28 460**	**239 523.3**

The estimated cost for the total project is about $240 000, maintenance and operation costs not included. The estimation has an accuracy of 50% and can, therefore, differentiate largely from the final costs.

Donors can partially pay the initial costs, however, when the system is built, it needs a sustainable payment system to pay for maintenance and operation costs, which is dependent on the inhabitants of Ward 10 if they are willing to pay for their water or if they find other means to collect water better. A continuous payment system by the users needs to be developed. Therefore, further surveys are required regarding the economical aspect, how to pay for the initial costs and maintain a sustainable system operation.

4.7 Management

Previously, the management and maintenance of the water system has been entirely done through volunteer work. A continuous payment fee would support a more sustainable management. The water committee should cooperate with local and national governments, state-owned utilities, possibly NGOs and donor agencies to start the project. (WHO, 2017) The system is designed to use locally available technologies that can be repaired and maintained by locals. A water committee with organized responsibilities will work more efficiently. A proposed set of responsibilities was set in the report in 2017 (Awais, et al., 2017):

- Coordination with the authorities and other departments
- Routine maintenance and inspections of the water systems
    - Routinized water analyses, especially CDBPs and pathogen analyses
    - Routinized cleaning and disinfection
- Resolving of eventual issues and repairs for users
    - Technical skills required
- Collecting of service fees
- Reporting of information to users

More detailed management positions are required to operate and maintain the network.

5 Results and discussion

The topography in Ward 10, combined with the rich water sources in the area, allows for a gravitationally driven water distribution system to be applied in the area. However, due to the topography of the terrain and nature of gravity-fed systems, one network only cannot be constructed to supply the whole of Ward 10 but must be divided into several separate smaller systems. For example, Line 8 and 9 supply water only to Nyabadza and Mutsamaba and is not being connected to the larger networks due to the large mountains between the systems. Additionally, lines 5 to 7 supply water to the majority of village cluster 1. Therefore, each of the systems can be reviewed individually, whether the population and water demand are large enough for a pipeline network to be constructed and the inhabitants are willing to pay to use a pipelined distribution system or there are other means of collecting water more suitable to the smaller villages. The decision must be made by the end users themselves through surveys, whereas the thesis provides the technical basis of the pipeline construction.

Before building a new system, the reasons for the old system's deterioration should be analyzed; otherwise there is a great risk of a new system sharing the same fate. According to the 2017 report, the biggest reason for the deterioration was payment failure, which led to mismanagement and ultimately caused the system's decay. The management system needs to be clearly setup and properly organized before the system is built. A continuous payment system needs to be settled as well to maintain and operate the network when the system is in place and the donors are out of the picture.

The project's success criteria are analyzed with a risk analysis (Table 35) and SWOT analysis (Table 36). The analyses are based on the previous report from 2017 but with a perspective from a gravitationally fed system rather than a general water supply harmonization. The risk analysis reviews potential risks and how to prevent or handle certain risks, were they to occur. The preventive measures reduce the possibility of the risks, and the containment measures reduce the impact if the potential risk occurs.

Table 35, risk analysis (Awais, et al., 2017)

Index	Probability	Impact
1	Rare	Insignificant
2	Unlikely	Minor
3	Possible	Moderate
4	Likely	Major
5	Almost certain	Catastrophic

Potential risk	Probability	Impact	Risk	Preventive measures	Containment measures
Lack of interest of usage towards payment system	5	4	20	Suitable pricing	Provide reassured safe drinking water
Lack of interest in installing treatment facilities	5	3	15	Education of potential hazards	Continuous water analyses
Breaking of a pipe	2	4	8	Routine maintenance and inspection	Supply of extra components
Unequal accessibility	3	4	12	Include end-users in planning and conduct more detailed planning	On-site measurements and surveys
Lack of maintenance	3	4	12	Payment and management	Organize and coordinate the team
Insufficient water supply	4	5	20	Additional intakes or larger storage tanks according to demand and supply patterns	Educate water use and irrigation styles

The SWOT analysis summarizes the strengths, weaknesses, opportunities, and threats graphically if a gravitationally fed distribution system were to be implemented to Ward 10. The analysis can help determine future decisions of the project.

Table 36, SWOT-analysis (Awais, et al., 2017)

Strengths	Weaknesses
• Simple technology • Locally available resources • Possibility to reuse parts of the old system • Optimal terrain for gravity-fed system	• Technical Assumptions need to be confirmed • Management and financial aspects need further surveys
Opportunities	**Threats**
• Partnerships with ZINWA and other authorities to manage the system • Possibility to provide jobs to locals	• Lack of interest from users and funders • The success of the project is dependent on the public opinion • Lack of management and maintenance • Overuse of water • No drainage of sullage

5.1 Further necessary technical data

The thesis provides rough dimensions and designs for a distribution network. Due to the proposal being made remotely, approximations and estimates have been used. Therefore, on site measurements are still needed, and additional data of the demand and supply are required to make more precise calculations and dimensions.

The demand and supply patterns are essential to dimensioning the systems components, especially sizes of the storage tanks. Too large water tanks are expensive to build, and unused water will stagnate, forming algae and CDBPs requiring unnecessary treatment. Contrarily, too small tanks will not meet the peak hour demands, and the users will be left without water. Further surveys need to be conducted of the water uses and habits in Ward 10 as well as analyses of the irrigation cycles to find a matching demand pattern to the area. Furthermore, education of the water use could be provided, for example, it is unnecessary to water the crops in the middle of the day. Currently, only a theoretical demand pattern has been used, which may not correlate with the actual pattern in Ward 10.

Water supply patterns are also needed, especially if surface water sources are used due to their varying seasonal capacity. Currently, only one measurement of the supply capacity has been conveyed during the dry season, which compels a minimal constant supply value to the dimensions. The capacities of the water sources should be measured throughout the year to measure the flow rates' difference to find a suitable supply pattern to the respective source. Due to only one supply measurement being available, multiple inlets are proposed to meet the supply and demand criterion. If a supply pattern is surveyed in the area over different seasons, suitable sized tanks can be dimensioned over longer time periods, which can reduce the number of intakes needed and, therefore, potentially also reducing the number of treatment facilities and maintenance required. The proposed intakes cannot guarantee a steady flow or supply, since the location is based on a topographical simulation only; they should be surveyed on site.

Potential groundwater sources such as spring catchments should also be further analyzed in the area, since they tend to provide a more reliable flow of water even during droughts. Also, the groundwater quality is less prone to pollution exposure than surface water. The water quality from the basic analysis tests seems to prove good quality, but further laboratory analyses of the water quality need to be analyzed to prove whether chlorination is treatment enough or whether additional treatment methods are needed.

The layout of the pipeline is determined from satellite view and follows certain infrastructure, and the elevation is determined with a LIDAR map. The demand for public houses such as schools, health centers, and business centers has not been taken into consideration. Due to their unknown location an average demand has been assumed. To confirm the accessibility of the pipeline layout, site-specific demands and pressures of the proposed system should be surveyed on site, and the layout can therefore change radically.

The next step in the project phase should be to conduct on site analyses of the water quality, intakes capacities and locations, pipelines layout, and water demands, as well as more details should be presented of the practical construction and installation of the components.

5.2 MIKE+ WD simulation results and discussion

MIKE+ Water Distribution is a commercial simulation program built on the demand-driven EPANET solver engine. The simulation on MIKE+ proved the theoretical functionality of the proposed system from a hydraulic perspective and, therefore, confirmed the dimensions of the pipes and capability of the demand flows.

Some issues with the software emerged when building the simulation network that limited the application to the results of the proposal. Adding storage tanks to the system interrupts the mass balance of the network. The net inflows of the storage tank, i.e. the inflows subtracted with the outflows of the tanks, are added to the total outflow of the system. According to the simulation, the net inflow is more often positive than negative, which means that more water is entering the tank than exiting, even though the tanks are set not to overflow. This issue raises the total water supplied to the system, therefore disrupting the total systems mass balance. However, by inserting strict rules in the simple controls, the overflow issue can be partially resolved with the following commands:

LINK 1 CLOSED IF NODE 2 ABOVE 20
LINK 1 OPEN IF NODE 2 BELOW 19

where LINK 1 is feeding water to NODE 2 which is a storage tank. These commands resolve the system's mass balance problem but add another unrealistic behavior to the network. With an infinite water reservoir as the supply, the refilling of the tanks happens instantly, which does not seem realistic in a gravity-pressurized system. Furthermore, when simulating larger networks with longer distribution lines and with multiple storage tanks, it causes sudden spikes in the total system's supply which do not correlate with the respective storage tank's refilling regarding volume and time step.

To overcome the issues with the solver, storage tanks have not been used in the simulation. Infinite reservoirs with a dummy node matching the total demand for the network the respective tank covers have been used as substitutes for storage tanks. The reservoir and dummy nodes' elevations are constant and equal to the surface elevation, which eliminates the dynamic water level which a realistic storage tank would have and, therefore, also assumes a constant minimal pressure in the succeeding pipelines. Due to the stiffness of the hard coded demand parameters set to each dummy node, no regulation or steering of the simulated network has been made. Only the hydraulic rules of the dimensions of the pipes were confirmed from the simulation results.

An attempt to use a different solver was made, with MIKE+ Collection Systems that models water drainages, but it was discarded due to lacking substitutes for a distribution network's components.

6 Conclusions

A technical framework of the functionality of a gravity-fed water distribution system was presented, where the theory of how pressure profiles and demand patterns determine the sizes and dimensions of the components in the system. Water quality, analyses, and treatment methods have been reviewed, as well as how they are included in a distribution system.

The presented theory was applied to Ward 10 Honde Valley, a village surveyed by a delegation from KTH and UPC in 2017. In the survey, basic data were gathered and provided by ZINWA, and the problem description of the water supply situation was evaluated. The main problem has been a deteriorating public distribution system, which has made villagers construct their own haphazardous water supply systems. The goal is to provide a technical basis for a public water distribution system that is controllably managed and maintained by authorities and supplies sufficient water to the inhabitants.

The area has been mapped with QGIS through satellite pictures, LIDAR, and interviews. Furthermore, the water sources were located by simulating a topographical simulation on QGIS to measure their sizes and potential uses to the distribution system. With this data and the data provided by the previous report as a basis, a hydrological simulation on MIKE+ WD has been done to verify the functionality of a possible distribution system to the area.

The results prove that, technically, a distribution system could be constructed in Ward 10, and rough dimensions of the components and costs have been presented. However, some further data gathering from the area would be beneficial to make a more detailed estimation of the network and the dimensions of its components. Thus, analyses of the water quality, intake capacities and locations, pipelines layout, water supply and demand patterns should be surveyed, and more details of the practical construction and installation of the components is necessary in successful continuation of the project.

Although, technically, Ward 10 suits a distribution system, many other aspects are vital to investigate. Is there a willingness to use the system compared to other means of collecting water that the area already uses, especially in the smaller villages where the demands are modest can the payment of a public system be too significant for the individual user? Clear management roles of a continuous operation for the system are vital for the system not to let it deteriorate, which requires continuous payment systems once the initial investment is done.

7 Swedish summary – Svensk sammanfattning

Projektförslag för ett gravitationsdrivet vattendistributionssystem till Honde Valley Ward 10

Detta examensarbete är en del av ett större utvecklingsprogram för Honde Valley Ward 10, Zimbabwe. År 2017 utfördes en exkursion till byn där en delegation bestående av studenter från KTH och UPC identifierade utvecklingspotential inom flera områden. Områdena var följande:

- vattendistribution, sanitet och rening
- hållbar energitillgång
- väg och infrastrukturförbättringar
- jordbruk
- samhällstjänster
- ekoturism

Vattendistributionen och reningen prioriterades som första område att utveckla. Lokalbefolkningen livnär sig på export av jordbruk och därmed har området behov av distribution av stora mängder vatten. Tillgången till rent och säkert dricksvatten för hushållsbruk och hälsostationer är bristfällig i och med avsaknaden av reningsanläggningar och omedvetenheten om riskerna av vattenburna sjukdomar. Enligt undersökningen från 2017 har 88 % av invånarna i Ward 10 klagat på att vattentillförseln är otillräcklig, och endast 26 % har åtkomst till det existerande distributionsnätverket. Resten av invånarna använder sig av brunnar eller andra tillvägagångssätt för att tillförse sig med vatten.

År 2003 byggdes ett vattendistributionsnätverk i området. Några år efter byggandet av rörledningssystemet överfördes ansvaret för förvaltning och underhåll från ZINWA till den lokala Samaringa-gemenskapskommittén, på grund av problem med betalning av vattenräkningarna. Därefter blev vattenförsörjningen avgiftsfri och underhållet utfördes av frivilliga. Efter ansvarsöverföringen började nätverket förfalla och förtroendet för den allmänna vattenförsörjningen sjönk. På grund av detta började lokalbefolkningen bygga sina egna rudimentära vattenledningssystem för sina egna behov. Dessa egenhändigt utförda system är oorganiserade och dränerar vattenresurserna mer än nödvändigt samt blir lämnade i miljön vartefter de börjar läcka och slutar att fungera. Vattenreningen för de egenhändigt gjorda systemen och det gamla allmänna systemet är obefintlig, konsumenterna själva ansvarar för att rena sitt dricksvatten.

Avhandlingens syfte är att ge en grov teknisk bakgrund för återuppbyggnaden av ett nytt allmänt vattendistributionssystem med inkluderad vattenrening som skulle kompensera för de oorganiserade egenhändigt gjorda systemen. För att komma upp till en tillräcklig kvantitet och kvalitet med tanke på hushållsbehoven och bevattningsvattnen krävs ett rörledningsdistributionsnätverk som är tillräckligt utbrett, och därtill bör sociologiska aspekter från lokalbefolkningens synpunkt tas i beaktande för att inte låta ett nytt system förfalla igen.

Avhandlingen är ett teoretiskt ramverk som anger hur ett gravitationsdrivet vattendistributionsnätverk i lantbyggsområden i utvecklingsländer kan byggas upp, när resurserna som används bör vara lokala. I ramverket ingår komponenterna som ett distributionssystem uppbyggs av samt hur dessa kan dimensioneras grovt. Komponenter som ingår i ett gravitationsdrivet distributionssystem är vattenupptagningspunkter, samlingstankar, lagringstankar, tryckreduceringstankar, rörledningar samt vattenreningsmetoder som långsamma sandfilter och klorering av systemet. Komponenternas dimensioneringar beror på behoven och tillgångarnas flödesvariation och kapacitet.

Teorin om distributionsnätverket tillämpas på situationen i Ward 10 och analyseras med ett simuleringsprogram, MIKE+. Topografiinformationen som simuleringens rörledningsnätverk baserar sig på är från en kartuppbyggnad som är ritad på QGIS utifrån satellitbilder, LIDAR-kartor och intervjuer med lokala. Med dessa informationskällor har hushållen och vattenresurserna lokaliserats samt höjdkurvorna och distanserna av rörledningarna definierats i Ward 10.

Distributionssystemet levererar 80 liter vatten per person per dag vilket enligt WHO:s standarder inkluderar vattenförbrukning för hushållsbehov samt bevattning för jordbruksbehov. Invånarantalet har beaktats med en 30 års perspektiv med 1,48 % befolkningstillväxt, vilket uppskattas till 5645 invånare. Det totala vattenbehovet har multiplicerats med ett dagligt variationsmönster för att beakta flödesförändringarna i systemet. Dessutom har generella tilläggskonstanter till flödet multiplicerats med grundbehovet för att kompensera för olika säsongsvariationer och övriga oförutsedda förbrukningar för att få det maximala designflödet i rörledningarna.

För att hitta tillräckliga vattenkällor för behoven har en topografisk simulering av regnvattenavledningen i området gjorts och kompletterats med resultat från en tidigare utförd grundvattenssimulering för att mäta vattenkällornas kapaciteter. Intagen till systemen har valts utifrån simuleringens resultat, varefter lokala mätningar har gjorts under torrperioden för att få konkreta flödesmätningar av bifloderna. De slutliga positionerna för intagen bör bestämmas på plats, där en tillräcklig höjdnivå för systemets tryck balanseras med tillräcklig kapacitet i vattenkällan oberoende årstid.

Eftersom dimensioneringarna på vattentankarna baserar sig på flödesvariationerna som har antagit teoretiska värden under ett dygn är dessa dimensioner bara riktgivande. Fler undersökningar bör göras i området om hur vattentillgångarnas flöden varierar med årstiderna samt hur vattenförbrukningen varierar under längre tider för att få ett mera realistiskt flödesvariationsmönster som beskriver en utsträckt tidsperiod och lämpar sig specifikt för Ward 10.

Det föreslagna distributionssystemet är ett integrerat system med två olika vattenbehov, bevattningsvatten och dricksvatten. Vattenrening rekommenderas för att inte låta dricksvattenreningen ske på invånarnas eget ansvar eftersom intagen till systemet är från ytvatten som lätt kontamineras. Däremot är rening av bevattningsvatten inte nödvändigt vilket kan medföra långsiktiga extra kostnader när vatten för bägge behoven renas i ett integrerat system. Alternativt kunde parallella system byggas för respektive vattenkvalitet men det dubblerar materialkostnaderna för konstruktionen.

Arbetet behandlar inte vilket tillvägagångssätt som lämpar sig bäst för Ward 10, utan är ett förslag som utgör grunden för projektet som vartefter får ta form av de lokala bestämmelserna och behoven.

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9 Appendices

9.1 Appendix A, Base data for Ward 10

NN	Name	Functional Tanks	Tank capacity (m3)	Inhabitants	Predicted inhabitants	Households	Current inhabitant water demand for 80 l/c/d	Predicted inhabitant water demand for 80 l/c/d	Storage required per irrigation cycle (m3)
	HONDE VALLEY								
	WARD 10	12	2048	3633	5645	1957	290640	451615	471600

NN	Name	Functional Tanks	Tank capacity (m3)	Inhabitants	Predicted inhabitants	Households	Current inhabitant water usage. 80 l/c/d	Predicted inhabitant water usage. 80 l/c/d	Storage required per irrigation cycle (m3)
	MANYONHO CLUSTER 4								
1	Manyonho	2	455.3	311	483	234	24880	38660	68040
2	Nyabadza	1	51.89	153	237	287	12240	19019	38160
3	Mutsamba	1	250	219	340	172	17520	27223	42840
4+	Jombe primary and secondary school								
5+	Two small Business Centres								
	TOTAL	**4**	**757.19**	**683**	**1061**	**693**	**54640**	**84903**	**149040**

	NGWENDE CLUSTER 3								
NN	Name	Functional Tanks	Tank capacity (m3)	Inhabitants	Predicted inhabitants	Households	Current inhabitant water usage. 80 l/c/d	Predicted inhabitant water usage. 80 l/c/d	Storage required per irrigation cycle (m3)
1	Ngwende	1	153	341	529	113	27280	42389	31680
2	Mutetwa	1	340	289	449	100	23120	35925	32040
3+	Masare Business Centre + two churches								
4+	FDC carpentry factory + motor mechanic shop								
5+	Social Tourism Centre								
6+	Honde Mission Technology								
7+	Honde Mission Clinic								
	TOTAL	**2**	**493**	**630**	**978**	**213**	**50400**	**78314**	**63720**

	SAMARINGA CLUSTER 2								
NN	Name	Functional Tanks	Tank capacity (m3)	Inhabitants	Predicted inhabitants	Households	Current inhabitant water usage. 80 l/c/d	Predicted inhabitant water usage. 80 l/c/d	Storage required per irrigation cycle (m3)
1	Samaringa / Mutsaka	1	372.6	162	251	57	12960	20138	19440
2	Samaringa schools	2	337.3	159	247	59	12720	19765	19440
3	Baradza	1	33.2	174	270	56	13920	21629	18360
4	Hambira	1	39.3	191	296	63	15280	23743	20880
5+	Samaringa primary and high school								
6+	Samaringa Clinic								
7+	Muitsi Business Centre								
	TOTAL	**5**	**782.4**	**686**	**1065**	**235**	**54880**	**85276**	**78120**

MATINGO CLUSTER 1									
NN	Name	Functional Tanks	Tank capacity (m3)	Inhabitants	Predicted inhabitants	Households	Current inhabitant water usage. 80 l/c/d	Predicted inhabitant water usage. 80 l/c/d	Storage required per irrigation cycle (m3)
1	Matingo	1	16.13	889	1381	446	71120	110510	68760
2	Nyakabinga	0	0	322	500	56	25760	40027	20520
3	Chipunha	0	0	163	253	81	13040	20262	27720
4	Saruwaka	0	0	260	404	172	20800	32320	63720
5	Gowa	0	0	(included in Saruwaka)		61			
6+	Small business center								
7+	Two preschools								
	TOTAL	**1**	**16.13**	**1634**	**2539**	**816**	**130720**	**203121**	**180720**

Irrigation		
Number of irrigation cycles	48	
Irrigation time	6	months
Irrigation once every	72	h

Population growth parameters		
Growth rate	1.48	%
Time span	30	years

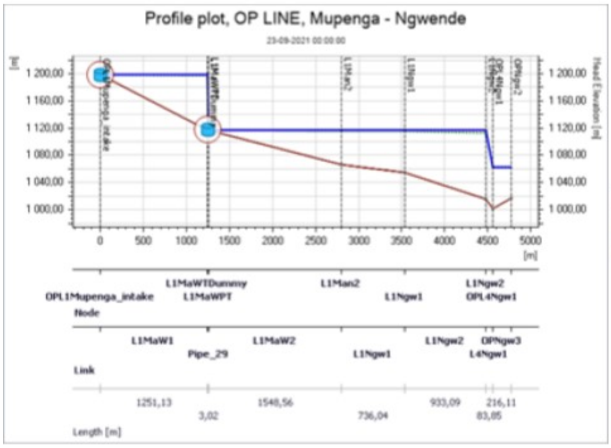
Demand	Current inhabitant ratio	Predicted inhabitant ratio
total LPCD	80	80
Irrigation LPCD	43.2	27.8
domestic LPCD	36.7	52.1
total LPS	3.36	5.22
Irrigation LPS	1.81	1.81
Domestic LPS	1.54	3.40

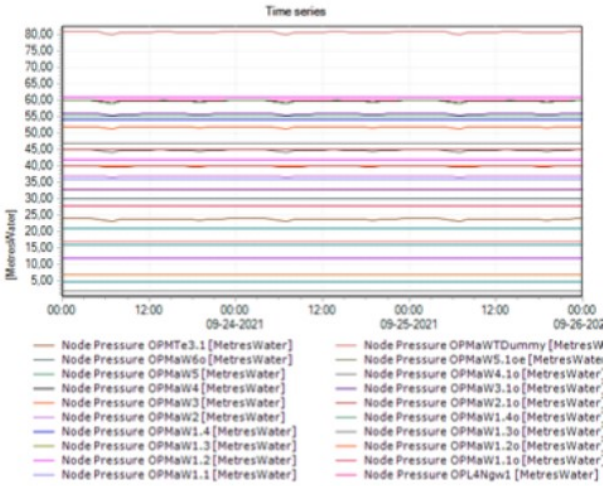
Multiplier data

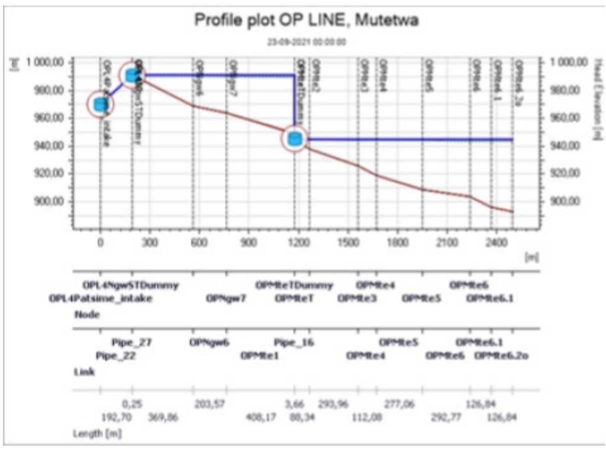
Time Period (h)	**Flow (unitless)**	**Multiplier**
1	0.2	0.491803
2	0.15	0.368852
3	0.12	0.295082
4	0.2	0.491803
5	0.4	0.983607
6	0.6	1.47541
7	0.7	1.721311
8	0.8	1.967213
9	0.63	1.54918
10	0.5	1.229508
11	0.42	1.032787
12	0.37	0.909836
13	0.28	0.688525
14	0.3	0.737705
15	0.3	0.737705
16	0.4	0.983607
17	0.48	1.180328
18	0.58	1.42623
19	0.7	1.721311
20	0.6	1.47541
21	0.44	1.081967
22	0.3	0.737705
23	0.24	0.590164
24	0.05	0.122951
avg.	0.406667	1

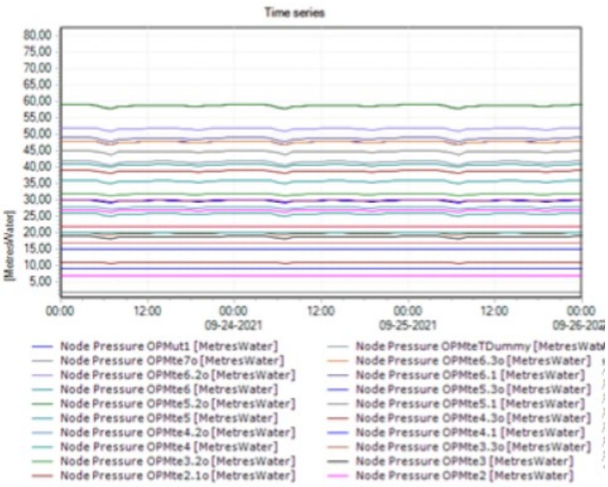
9.2 Appendix B, Pressure profiles and pressures in pipelines

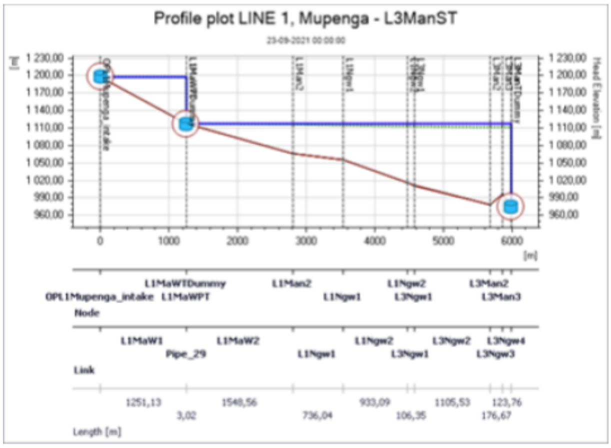
It is strived to keep a minimum pressure of 1 bar in the system and pressure between 1-3 bar at each outlet. Nodes under 1 bar are usually next to a water tank which according to the simulation is empty, but in reality does have a pressure if the tank is containing water. Nodes above 3 bar are mostly junction nodes or dummy nodes, therefore requiring a higher PN pipe for that section.

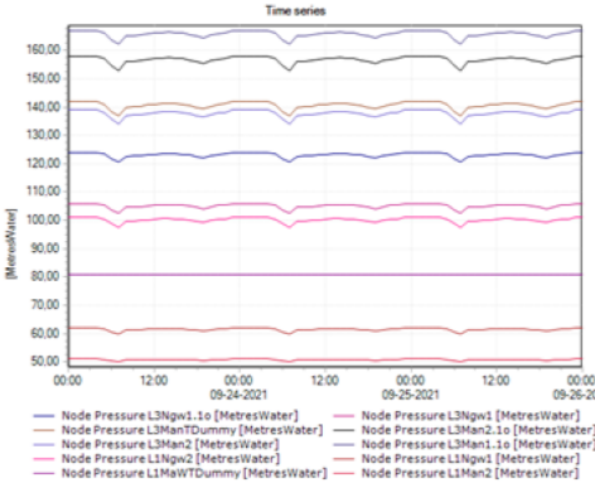









Profile plot LINE 1, Mupenga - L3ManST
23-09-2021 00:00:00
Node
Link
Length [m]


Time series
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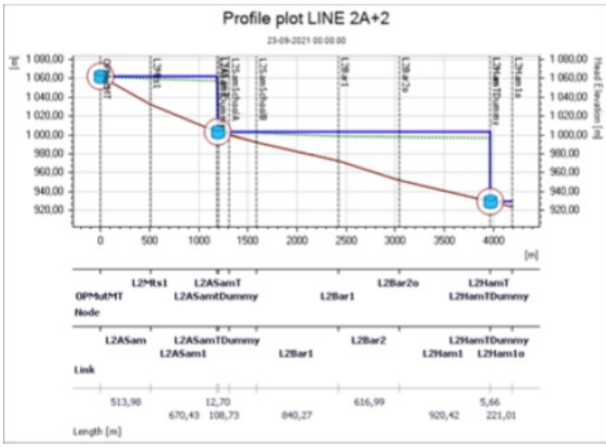
Node Pressure L3Ngw1.1o [MetresWater]
Node Pressure L3Ngw1 [MetresWater]
Node Pressure L3ManTDummy [MetresWater]
Node Pressure L3Man2.1o [MetresWater]
Node Pressure L3Man2 [MetresWater]
Node Pressure L3Man1.1o [MetresWater]
Node Pressure L1Ngw2 [MetresWater]
Node Pressure L1Ngw1 [MetresWater]
Node Pressure L1MaWTDummy [MetresWater]
Node Pressure L1Man2 [MetresWater]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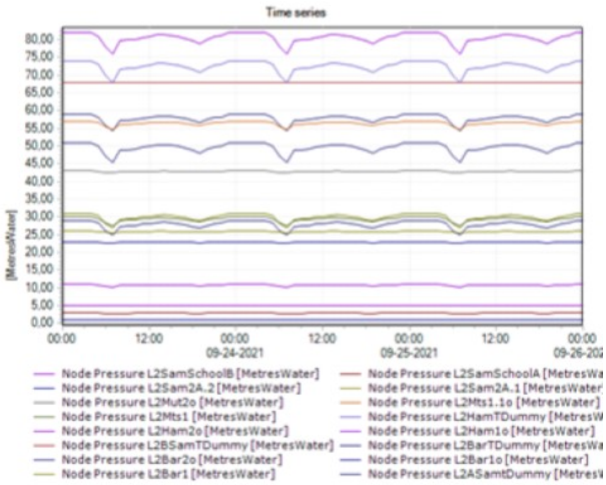
Node
Link
Length [m]


Time series
[MetresWater]
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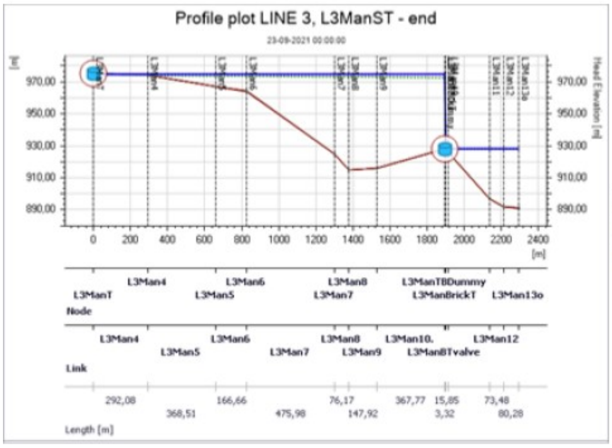
Node Pressure L2SamSchoolA [MetresWa
Node Pressure L2Sam2A.2 [MetresWater]
Node Pressure L2Sam2A.1 [MetresWater
Node Pressure L2Mut2o [MetresWater]
Node Pressure L2Mts1.1o [MetresWater]
Node Pressure L2Mts1 [MetresWater]
Node Pressure L2HamTDummy [MetresW
Node Pressure L2Ham2o [MetresWater]
Node Pressure L2Ham1o [MetresWater]
Node Pressure L2BSamTDummy [MetresWater]
Node Pressure L2BarTDummy [MetresWa
Node Pressure L2Bar2o [MetresWater]
Node Pressure L2Bar1o [MetresWater]
Node Pressure L2Bar1 [MetresWater]
Node Pressure L2ASamtDummy [MetresW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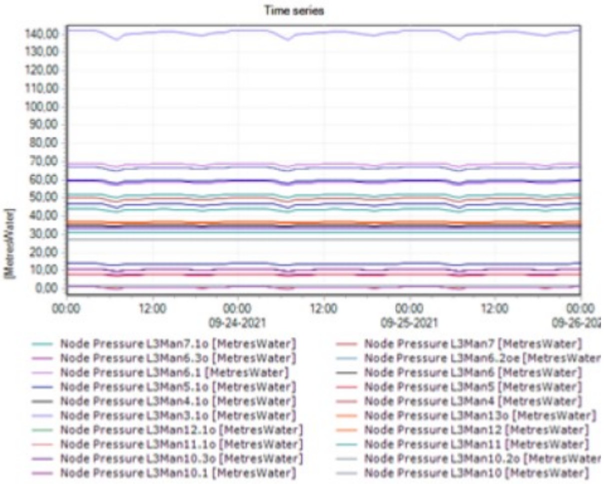
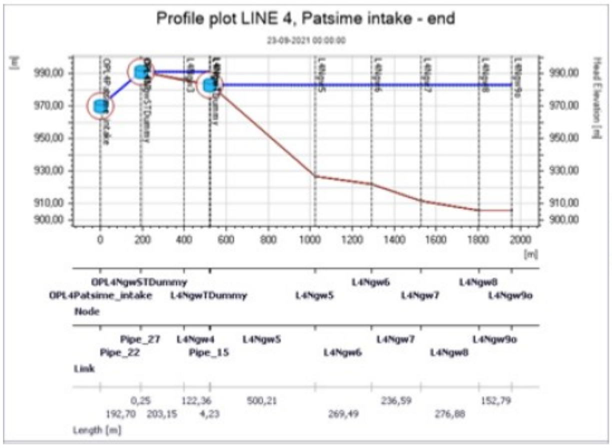

Time series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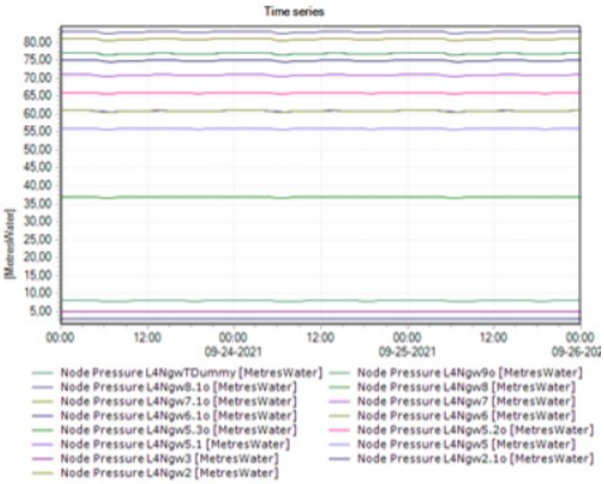
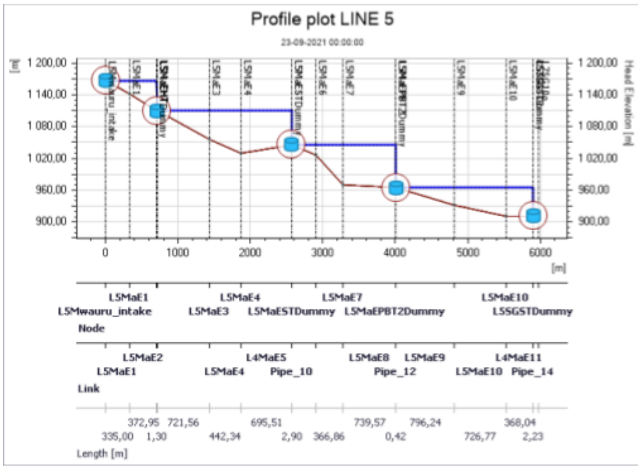

Time series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Length [m]
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Link
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L5Mwauru_intake
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L5MaE4
L5MaESTDummy
L5MaE7
L5MaEPBT2Dummy
L5MaE10
L5SGSTDummy
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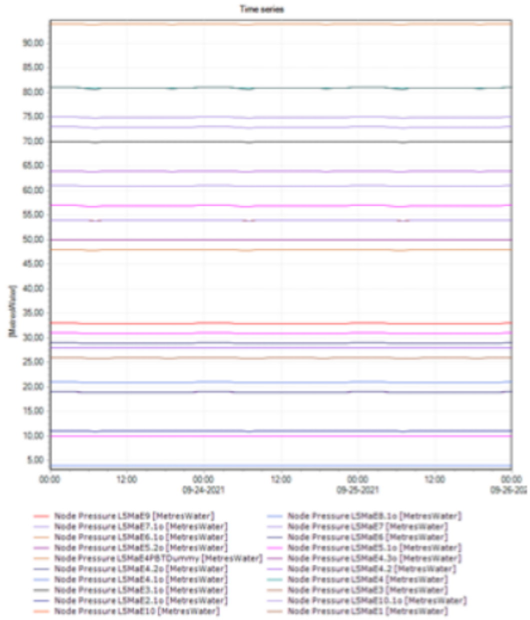
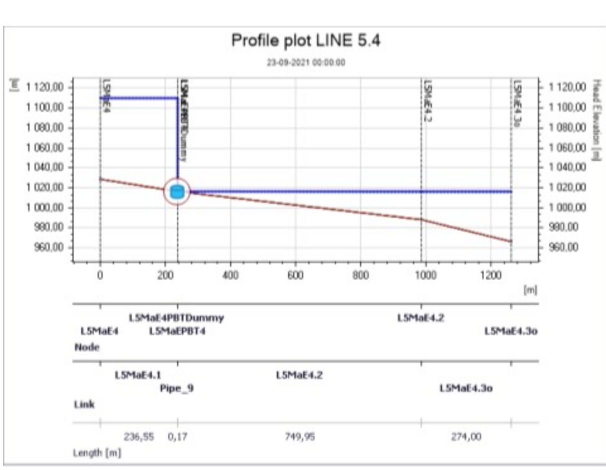
L5MaE10
Pipe_14


Time series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Length [m]
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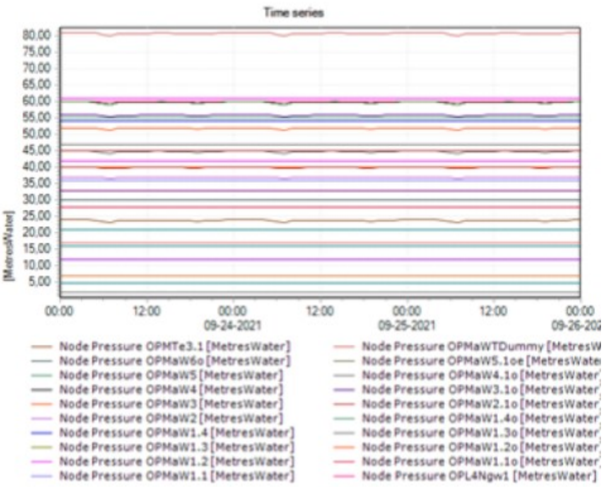
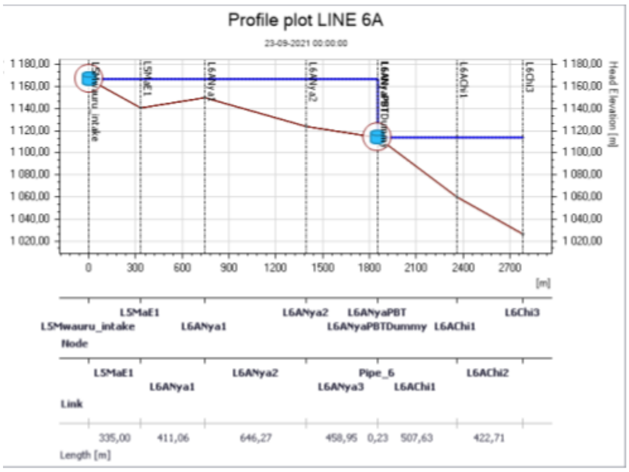
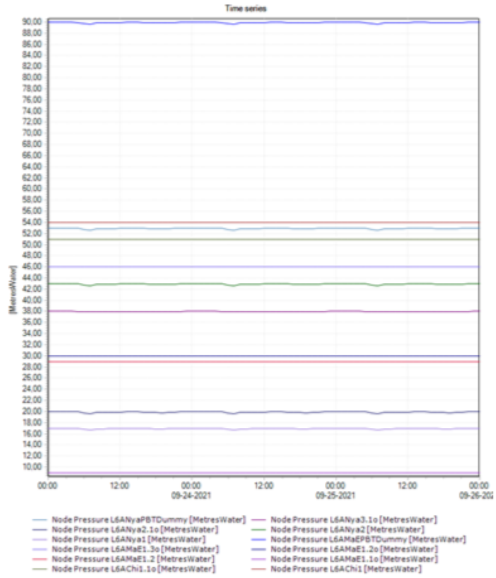
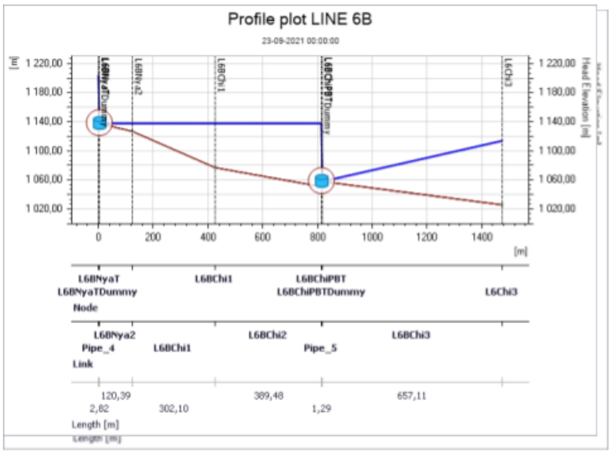
L5MaE4.3o
L5MaE4.1
Pipe_9
236,55
0,17
749,95
274,00

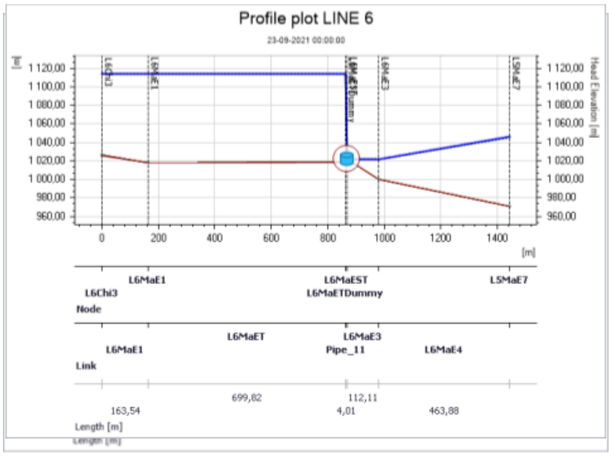

Time series
[MetresWater]
09-24-2021
09-25-2021
Node Pressure OPMTe3.1 [MetresWater]
Node Pressure OPMaW6o [MetresWater]
Node Pressure OPMaW5 [MetresWater]
Node Pressure OPMaW4 [MetresWater]
Node Pressure OPMaW3 [MetresWater]
Node Pressure OPMaW2 [MetresWater]
Node Pressure OPMaW1.4 [MetresWater]
Node Pressure OPMaW1.3 [MetresWater]
Node Pressure OPMaW1.2 [MetresWater]
Node Pressure OPMaW1.1 [MetresWater]
Node Pressure OPMaW4.1o [MetresWater]
Node Pressure OPMaW3.1o [MetresWater]
Node Pressure OPMaW2.1o [MetresWater]
Node Pressure OPMaW1.4o [MetresWater]
Node Pressure OPMaW1.3o [MetresWater]
Node Pressure OPMaW1.2o [MetresWater]
Node Pressure OPMaW1.1o [MetresWater]
Node Pressure OPL4Ngw1 [MetresWater]

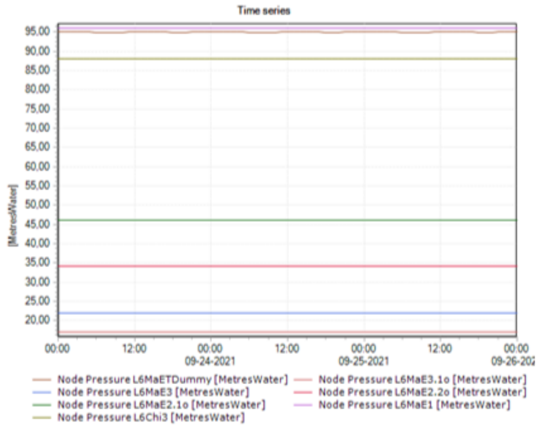

Profile plot LINE 6A
23-09-2021 00:00:00
Head Elevation [m]
[m]
L5MaE1
L5Mwauru_intake
L6ANya1
L6ANya2
L6ANyaPBT
L6ANyaPBTDummy
L6AChi1
L6Chi3
Node
L5MaE1
L6ANya1
L6ANya2
L6ANya3
Pipe_6
L6AChi1
L6AChi2
Link
335,00
411,06
646,27
458,95
0,23
507,63
422,71
Length [m]

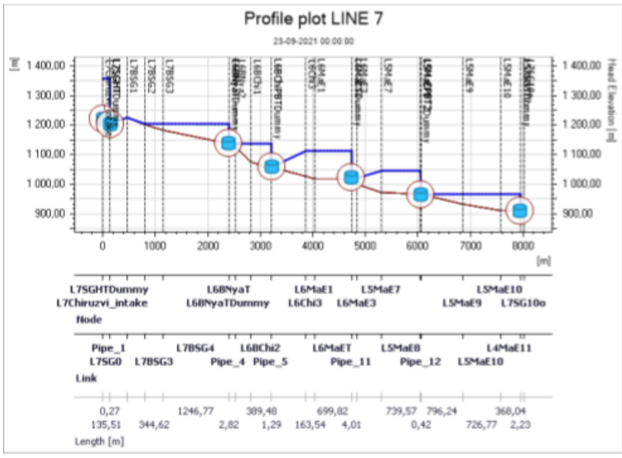

Time series
[MetresWater]
09-24-2021
09-25-2021
Node Pressure L6ANyaPBTDummy [MetresWater]
Node Pressure L6ANya3.1o [MetresWater]
Node Pressure L6ANya2.1o [MetresWater]
Node Pressure L6ANya2 [MetresWater]
Node Pressure L6ANya1 [MetresWater]
Node Pressure L6AMaEPBTDummy [MetresWater]
Node Pressure L6AMaE1.3o [MetresWater]
Node Pressure L6AMaE1.2o [MetresWater]
Node Pressure L6AMaE1.2 [MetresWater]
Node Pressure L6AMaE1.1o [MetresWater]
Node Pressure L6AChi1.1o [MetresWater]
Node Pressure L6AChi1 [MetresWater]

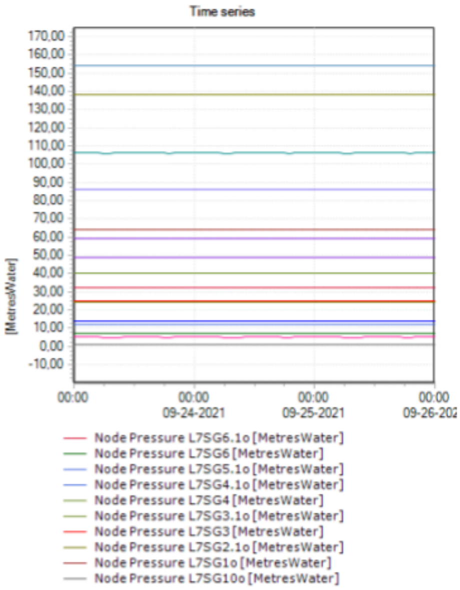

Profile plot LINE 6B
23-09-2021 00:00:00
Head Elevation [m]
[m]
L6BNyaT
L6BNyaTDummy
L6BChi1
L6BChiPBT
L6BChiPBTDummy
L6Chi3
Node
L6BNya2
Pipe_4
L6BChi1
L6BChi2
Pipe_5
L6BChi3
Link
120,39
2,82
302,10
389,48
1,29
657,11
Length [m]

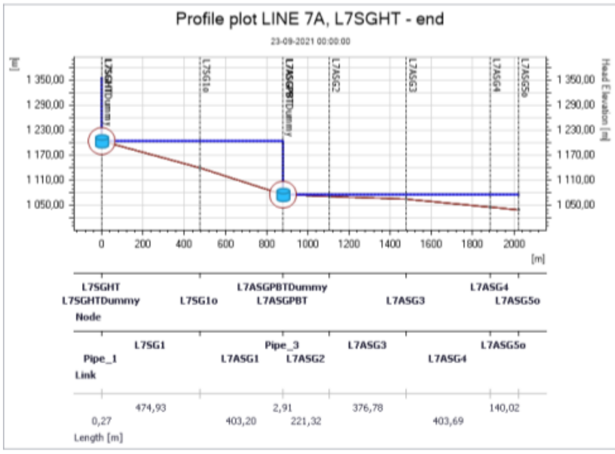

Time series
[MetresWater]
09-24-2021
09-25-2021
Node Pressure L6BNyaTDummy [MetresWater]
Node Pressure L6BNya2.1o [MetresWater]
Node Pressure L6BNya2 [MetresWater]
Node Pressure L6BNya1o [MetresWater]
Node Pressure L6BChiPBTDummy [MetresWater]
Node Pressure L6BChi2.1o [MetresWater]
Node Pressure L6BChi1.1o [MetresWater]
Node Pressure L6BChi1 [MetresWater]


Profile plot LINE 6
23-09-2021 00:00:00
Node
Link
Length [m]


Time series
[MetresWater]
Node Pressure L6MaETDummy [MetresWater]
Node Pressure L6MaE3 [MetresWater]
Node Pressure L6MaE2.1o [MetresWater]
Node Pressure L6Chi3 [MetresWater]
Node Pressure L6MaE3.1o [MetresWater]
Node Pressure L6MaE2.2o [MetresWater]
Node Pressure L6MaE1 [MetresWater]

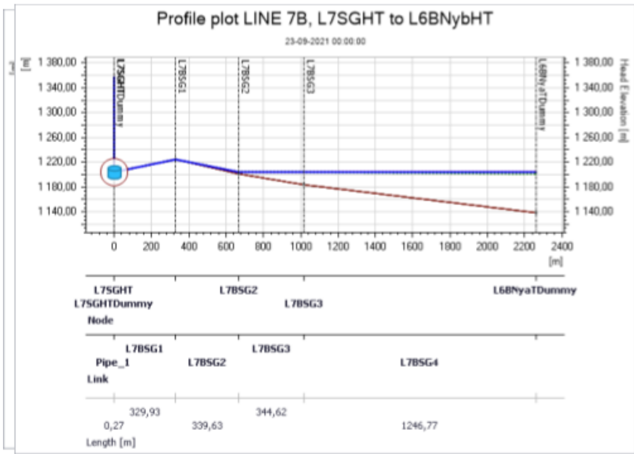

Profile plot LINE 7
23-09-2021 00:00:00
Node
Link
Length [m]


Time series
[MetresWater]
Node Pressure L7SG6.1o [MetresWater]
Node Pressure L7SG6 [MetresWater]
Node Pressure L7SG5.1o [MetresWater]
Node Pressure L7SG4.1o [MetresWater]
Node Pressure L7SG4 [MetresWater]
Node Pressure L7SG3.1o [MetresWater]
Node Pressure L7SG3 [MetresWater]
Node Pressure L7SG2.1o [MetresWater]
Node Pressure L7SG1o [MetresWater]
Node Pressure L7SG10o [MetresWater]

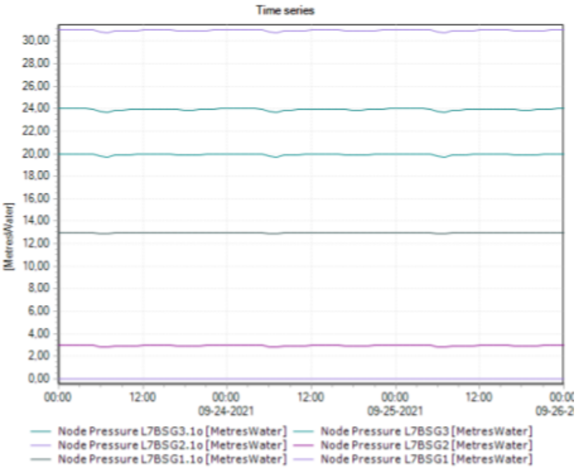


Profile plot LINE 7A, L7SGHT - end
23-09-2021 00:00:00
Head Elevation [m]
Length [m]

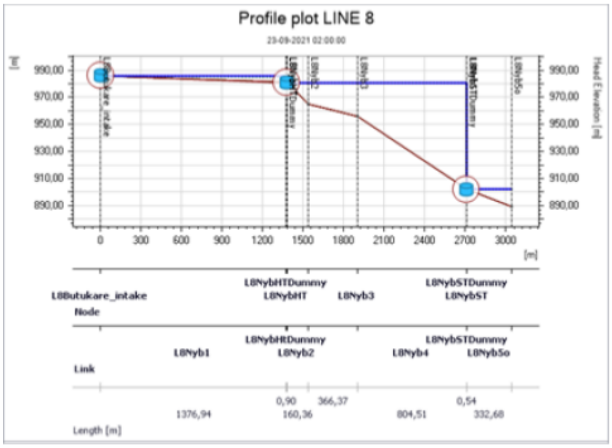
Time series
[MetresWater]
Node Pressure L7ASGPBTDummy [MetresWater]
Node Pressure L7ASG5o [MetresWater]
Node Pressure L7ASG4.1o [MetresWater]
Node Pressure L7ASG4 [MetresWater]
Node Pressure L7ASG3.1o [MetresWater]
Node Pressure L7ASG3 [MetresWater]
Node Pressure L7ASG2.1o [MetresWater]
Node Pressure L7ASG2 [MetresWater]

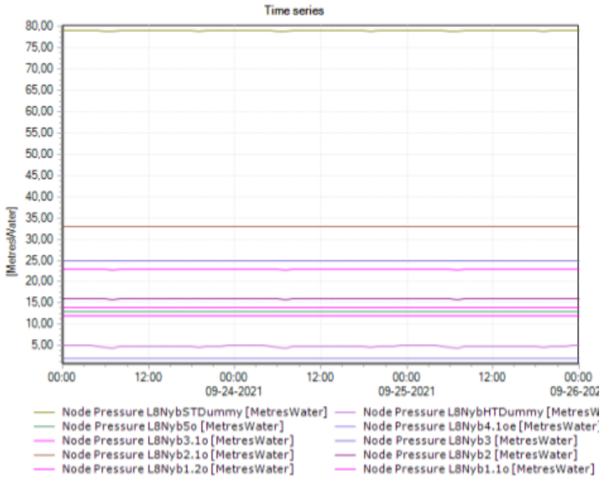


Profile plot LINE 7B, L7SGHT to L6BNybHT
23-09-2021 00:00:00
Head Elevation [m]
Length [m]

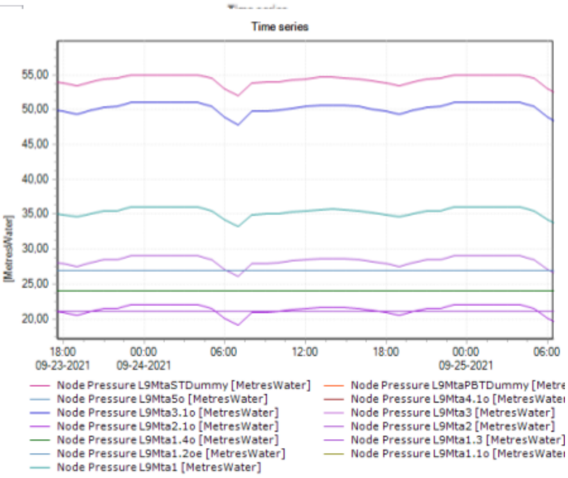


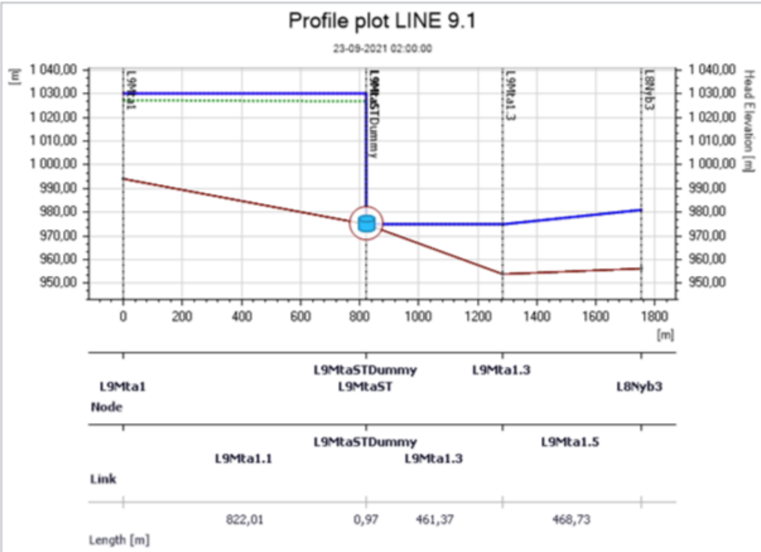
Time series
[MetresWater]
Node Pressure L7BSG3.1o [MetresWater]
Node Pressure L7BSG3 [MetresWater]
Node Pressure L7BSG2.1o [MetresWater]
Node Pressure L7BSG2 [MetresWater]
Node Pressure L7BSG1.1o [MetresWater]
Node Pressure L7BSG1 [MetresWater]


Profile plot LINE 8
23-09-2021 02:00:00
L8Butukare_intake
L8NybHTDummy
L8NybHT
L8Nyb3
L8NybSTDummy
L8NybST
Node
L8Nyb1
L8NybHTDummy
L8Nyb2
L8Nyb4
L8NybSTDummy
L8Nyb5o
Link
1376,94
0,90
160,36
366,37
804,51
0,54
332,68
Length [m]


Time series
[MetresWater]
Node Pressure L8NybSTDummy [MetresWater]
Node Pressure L8Nyb5o [MetresWater]
Node Pressure L8Nyb3.1o [MetresWater]
Node Pressure L8Nyb2.1o [MetresWater]
Node Pressure L8Nyb1.2o [MetresWater]
Node Pressure L8Nyb4.1oe [MetresWater]
Node Pressure L8Nyb3 [MetresWater]
Node Pressure L8Nyb2 [MetresWater]
Node Pressure L8Nyb1.1o [MetresWater]


Profile plot LINE 9
23-09-2021 02:00:00
L9Nyamarungu_intake
L9Mta1
L9Mta2
L9Mta3
L9MtaPBT
L9MtaPBTDummy
L9Mta5o
Node
L9Mta1
L9Mta2
L9Mta3
L9MtaPBTDummy
L9Mta4
L9Mta5
Link
855,67
134,38
528,11
382,14
0,25
540,35
Length [m]


Time series
[MetresWater]
Node Pressure L9MtaSTDummy [MetresWater]
Node Pressure L9Mta5o [MetresWater]
Node Pressure L9Mta3.1o [MetresWater]
Node Pressure L9Mta2.1o [MetresWater]
Node Pressure L9Mta1.4o [MetresWater]
Node Pressure L9Mta1.2oe [MetresWater]
Node Pressure L9Mta1 [MetresWater]
Node Pressure L9Mta3 [MetresWater]
Node Pressure L9Mta2 [MetresWater]
Node Pressure L9Mta1.3 [MetresWater]



Profile plot LINE 9.1
23-09-2021 02:00:00
[m]
Head Elevation [m]
L9Mta1
L9MtaSTDummy
L9Mta1.3
L8Nyb3
L9MtaSTDummy
L9Mta1.3
L9Mta1
L9MtaST
L8Nyb3
Node
L9MtaSTDummy
L9Mta1.5
L9Mta1.1
L9Mta1.3
Link
822,01
0,97
461,37
468,73
Length [m]