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2024-02

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Magabatela, Jonathan. (2024). Hydraulic Performance Assessment and Upgrading of The Tswelopele Village (Gauteng) Water Distribution Network. [Master's dissertation, University of the Witwatersrand]. WIReDSpace. https://hdl.handle.net/10539/38887

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HYDRAULIC PERFORMANCE ASSESSMENT AND UPGRADING OF THE TSWELOPELE VILLAGE (GAUTENG) WATER DISTRIBUTION NETWORK.



Jonathan Magabatela

A research report submitted to the Faculty of Engineering and the Built Environment, University of the Witwatersrand, Johannesburg, in partial fulfilment of the requirements for the degree of Master of Science in Engineering.

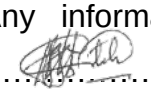
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Johannesburg, 2024

DECLARATION

I declare that this research report is my own unaided work. It is being submitted for the Degree of Master of Science in Engineering to the University of the Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination to any other University.

Any information used in this Research Report has been obtained by me.

…………

14th day of February 2024

ABSTRACT

The hydraulic performance assessment of a water distribution network can reveal critical information that might help to address challenges experienced within that network. This study investigated the hydraulic performance and upgrading of the Tswelopele water distribution network to accommodate additional 35 yard connections. In line with the main objective of the study, the following were addressed, (1) modelling and simulating in EPANET the existing Tswelopele water distribution network and ascertaining the nodes experiencing below minimum pressures, (2) determining and incorporating upgrades required to accommodate current and future water demands, (3) optimization of the proposed upgraded water distribution network, initially using trial and error typically employed by industry and then using Wadiso software and (4) comparison of results obtained using the two approaches. Tswelopele Village comprises 89 households, and only 54 are connected to the existing water distribution network. The existing water distribution network was refurbished in 2015 but some households which are part of the water distribution network are still experiencing erratic water supply. A model of the existing network was prepared and simulated in EPANET. The analysis results indicated that only one node had a pressure value below the normally minimum accepted of 20m, but it was more than the minimum allowable pressure of 15m. Therefore, it is suspected that the water challenges might not be due to low pressure but to other reasons such as leakages, partially open valves or illegal connections. To accommodate the additional 35 households, an upgrade of the network incorporating future water demands was proposed. Optimization by trial and error was employed as per usual industry approach and was checked by optimization with Wadiso software. A comparison of the results revealed that based on July 2023 pipe cost estimates a 19.5% saving could be realised by using Wadiso optimization software as compared to the traditional trial and error.

DEDICATION

This research report is dedicated to my beloved wife Cresencia, our sons Kevin Blessings and Kaiden Alexis, and my late parents, Joshua Magabatela and Siphiwe Magabatela, nee Sithole.

ACKNOWLEDGEMENTS

Firstly, I would like to thank Professor Adesola Ilemobade, my supervisor for his immense support and guidance.

Secondly, I would like to thank Dr Alexander Sinske and his team at GLS Software for their assistance with Wadiso student version.

I am also grateful for the assistance rendered by Mr Sello Semenya and his team at SFC Engineers (SFCE), the staff at the Department of Rural Development and Land Reform and also Mogale City Local Municipality.

I would especially like to thank my dearest wife, Cresencia, and our sons Kevin Blessings and Kaiden Alexis for their love, constant support, and encouragement.

Finally, all Glory and Honour to our Lord Jesus Christ. *You have made us for yourself, O Lord, and our hearts are restless until they rest in you*.

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LIST OF SYMBOLS

km	Kilometre
L	Litre
L/s	Litres per second
Lps	Litres per second
L/c/d	Litres per capita per day
L/d	Litres per day
m^3	Cubic metre
m^3/s	Cubic metre per second
m^3/h	Cubic metre per hour
m^3/d	Cubic metre per day
R	South African Rand

LIST OF ABBREVIATIONS AND ACRONYMS

ACCOL	Adaptive Cluster Covering with Local Search
AWWA	American Water Works Association
CRDP	Comprehensive Rural Development Programme
CRS	Controlled Random Search
DDA	Demand Driven Analysis
DHS	Department of Human Settlements
DP	Dynamic Programming
DRDLR	Department of Rural Development and Land Reform
DWAF	Department of Water Affairs and Forestry
EPS	Extended Period Simulation
fmincon	find minimum of constrained nonlinear multivariable function
HDPE	High Density Polyethylene
LP	Linear Programming
LPG	Linear Programming Gradient
NLP	Non-linear Programming
NLPG	Non-Linear Programming Gradient
NHFR	Node Head-Flow Relationship
PDA	Pressure Driven Analysis
SAMOCSA	Self-Adaptive Multi-Objective Cuckoo Search Algorithm
SANS	South African National Standards
SDG	Sustainable Development Goal
SMGA	Structured Messy Genetic Algorithm
SS	Steady State
UN	United Nations
Wadessy	WAter DEcision Support System
Wadiso	Water Distribution System Optimization Software
WDN	Water Distribution Network
WDS	Water Distribution System
WRC	Water Research Commission
WRN	Water Reticulation Network

1. INTRODUCTION

Every human life on earth requires water for survival. Without water, there can be no life as all organisms (including human beings) require water for sustenance. The scarcity of water negatively impacts plants and animal life. In July 2010, the sixty-fourth session of the United Nations (UN) General Assembly adopted resolution 64/292, which recognises “the right to safe and clean drinking water and sanitation as a human right that is essential for the full enjoyment of life and all human rights” (United Nations 2010:2). The UN Committee on Economic Social and Cultural Rights further stated that this human right seeks to make sure that sufficient, safe, acceptable, physically accessible, and affordable water is provided to all people for personal and domestic uses (United Nations 2010). Several member states have similarly followed suit and declared the right to water for their citizens. In South Africa, the basic human right to access sufficient water is enshrined in the Constitution and Bill of Rights and all local government authorities are obliged to provide free basic potable water to their local residents (DWAF 2001). However, despite these notable declarations, access to clean water is still a dream for many people. The United Nations estimates that as of 2020, two billion people worldwide did not have access to safely managed drinking water services (United Nations 2022). As part of its effort to address this problem, the General Assembly of the UN, at its meeting in New York City in 2015, included access to safe drinking water by 2030, as one of the 17 Sustainable Development Goals (SDGs) (United Nations 2015).

In a bid to reduce rural poverty, the South African government launched the Comprehensive Rural Development Programme (CRDP) in 2009. The CRDP sought to maximize the use and management of natural resources to create vibrant, equitable and sustainable rural communities (CRDP 2009). To mitigate the vulnerability of rural communities to hunger and to increase food security, the provision of water was one of the key initiatives of the programme. The Department of Rural Development and Land Reform (DRDLR) was subsequently tasked with the implementation of infrastructural development programmes aimed at achieving part of the strategic objectives of CRDP related to integrated development (CRDP 2009). These programmes mainly comprised of design and construction of water infrastructure that included pipelines, tanks, pumps, valves, and boreholes. The

programme grappled with several challenges such as political interference, tight budgets and timelines which then affected the delivery and quality of the projects. Maroo *et al*. (2015) highlighted that there was a need for political principals to allow for due process to be followed with regards to implementation of projects to avoid stretching the scope.

One village that benefited from this programme is the Tswelopele Village situated on the southern slopes of the Magaliesberg Mountain in Hekpoort, Mogale City Local Municipality. The existing water supply to the Village covers about 60% of the households (SFCE 2015). Before the CRDP rehabilitation project in 2015, the water supply to the Village was provided by an electric powered borehole located near the Village which was later decommissioned due to ever-increasing electrical costs and vandalism. The Village was then supplied with potable water from commercial sources by different service providers who were contracted by the Local Municipality (SFCE 2015). The rehabilitation focused on replacing the existing pipelines and standpipes without allowing for upgrading of the system or inclusion of new yard connections. This left the Village with a rehabilitated water system which was similar to the previous network and sadly, did not improve water supply to individual households (SFCE 2015). The same report by SFCE, indicated that there were cases of low pressure in some high lying areas in the water distribution network that were reported especially during peak times.

A water distribution network conveys water from the source or treatment facility to the users through a network of pipes, tanks, valves, and pumps. The amount of water required by users to satisfy their needs is known as the water consumption and it varies considerably during the day. It is expected that the water demand must be met and delivered at adequate pressure and flow. The performance of a water distribution network can be measured by its capacity to provide reliable quantities of water to users at acceptable pressures in an economical and sustainable way. Poor performance of the water distribution network can have an impact on water quality by increasing water age and promoting biofilm development. The poor performance of a water distribution network can be attributed to varied reasons including poor design, aging network, and pressure fluctuations.

The costs involved in the construction, maintenance and operation of a water distribution network is normally very large. The sizing of pipe diameters is one of the

most important design aspects of WDN because their cost normally caters for a considerable portion of the total costs (Vaidya and Mali 2019). Large diameter pipes mean higher capital costs. On the contrary, undersized pipes also present challenges because they may be affected by pipe head losses resulting in loss of pressure and the introduction of water hammer issues (DHS 2019). Therefore, the selection of pipes that deliver water at the required pressure at demand points at a lower cost becomes important in the design of rural water schemes.

The diameter of the pipes placed in the WDN and the type of WDN governs the problem of optimization. The network problem, particularly the optimal diameter allocation of pipes in a WDN, has been investigated by various researchers in the past (Sangroula *et al.* 2022). Several mathematical solutions and methods have been provided, which include Genetic Algorithms, Linear Programming (LP) and Non-Linear Programming (NLP) and these have been used over the years.

This research project seeks to investigate the difference in total pipe costs based on optimization by trial and error method and optimization software. This comparison will be performed after initially assessing the hydraulic performance of the existing network and upgrading it using EPANET. Rossman *et al.* (2020) describes EPANET as a software that performs both extended period simulation of hydraulic and water quality behaviour within pressurised pipe networks and it is also capable of perform steady state simulation. EPANET predicts the flow of water in a pipe distribution network and pressure variations at nodes during operations. Most designers typically employ trial and error method in determining network pipe sizes (Ilemobade and Stephenson 2003), and then simulate the network using EPANET. Trial and error involve testing several trial networks to select the most optimum design.

Given the inability of EPANET to optimally size pipe diameters, a local optimizer software was sought. Wadessy developed by Ilemobade (2023) and Wadiso (Water Distribution Simulation and Optimization) developed by GLS (GLS 2023) were two software that were suggested. However, Wadiso was used because a student version was freely available and functional. Wadiso is a software with its origins from the Colorado State University where it was developed by Prof. Johannes Gessler (GLS 2023) and was later developed for commercial purposes in South Africa by GLS. Initially, Wadessy software (Ilemobade and Stephenson 2003) was to be employed for this research. Wadessy is also a free software developed by

Ilemobade (2003) and was made available for this study. The main challenge with using Wadessy is that it was created using DELPHI in Windows 3.2 operating system almost 20 years ago and is no longer compatible with the current Windows 10 and 11 operating systems.

1.1 Problem Statement

Report shows some houses within the Tswelopele Village have been experiencing water supply challenges i.e., water delivered at low pressures to some houses and some houses not receiving water. This situation has been attributed to the poorly designed water distribution network (SFCE 2023). The increasing population of the Village (SFCE 2023) means that these problems will be exacerbated and experienced by more households.

Therefore, studying the existing water supply condition and upgrading the water distribution network, using forecasted future water demands, is crucial for addressing the challenges bedevilling the community. No research has been previously conducted on Tswelopele Village water distribution network. This study is thus necessary for aiding effective decision making on the upgrading of the existing water distribution network at minimal cost.

1.2 Aim and Objectives of the study

The aim of this research is to investigate the hydraulic performance and upgrading of the Tswelopele Village water distribution network. This will be achieved using the EPANET software and comparing two optimization methods – (i) the partial enumeration technique used by Wadiso and (ii) trial and error, typically employed by industry. This investigation will check if utilization of partial enumeration technique will reduce the total pipe cost from that obtained by trial and error.

1.2.1 Specific objectives

The aim of this research can be addressed in the following objectives:

i. To use an EPANET model to simulate the existing Tswelopele WDN and find (and/or verify) the nodes experiencing below minimum pressures and flows.
ii. To determine and incorporate into the Tswelopele WDN, the upgrades required to meet current and future water demands.

iii. To optimize the proposed upgrade to the Tswelopele WDN using trial and error typically employed by industry.
iv. To optimize the proposed upgrade to the Tswelopele WDN using the Wadiso software.
v. To compare the results obtained using trial and error with those obtained using Wadiso.

1.3 Significance of the study

This study seeks to highlight the shortcomings of trial and error in the design of optimal water distribution networks and find out if cost-effective water distribution networks can be designed using optimization software like Wadiso. Even though EPANET is a powerful tool for design and analysis of water distribution networks, it falls short when it comes to identifying the most efficient design in terms of pipe sizing. With escalating pipe costs, which in turn, affects water distribution costs, it is now imperative to search for the most efficient design that can deliver water to end users at optimum conditions.

1.4 Scope and Limitations of the study

This study will investigate the cost saving that can be realised by using Wadiso software in the optimization of WDN's in lieu of trial and error. The Tswelopele community water distribution network will be initially optimized using trial and error in combination with guidelines typically employed by industry. The results from this exercise will then be compared with the results from optimising the Tswelopele community water distribution network using Wadiso.

This study is limited to Tswelopele Village and will allow only new connections to households that are already part of Tswelopele Village (35 households) and will cater for the future demands of all the 89 households. The modelling effort will only focus on hydraulic modelling and not the water quality aspect due to the lack of data. A cost estimate of the upgraded network will be presented based on the prices as of July 2023.

Some of the problems encountered during the study include lack of adequate records on households and their water consumption records from the relevant authorities. There is also little information about the Village and population details. The latest census results could not be obtained from the Stats SA or Mogale City

Local Municipality. Therefore, the population figures were based on the household count and each household was assumed to comprise of 6 people.

1.5 Structure of the Research Report

This research report is structured as follows:

Chapter 1: Introduction. The problem of design and optimization of water distribution networks is introduced. The need for an optimum design is motivated and the chapter concludes by stating the objectives of the research, and significance, scope, and limitations of the study.

Chapter 2: Literature Review. The existing literature on the design of water distribution networks is explored. The use of EPANET as a design tool including discussion on Demand Driven Analysis (DDA), Pressure Driven Analysis (PDA), Steady State Simulation (SSS) and Extended Period Simulation (EPS) is presented. Also, the optimization techniques employed in water distribution systems design including trial and error, Wadessy and Wadiso are discussed in detail in this chapter.

Chapter 3: Case Studies of Simulation – Optimization of WRNs

A brief review of some of the popular benchmark networks used in case studies of various optimization techniques is presented. The benchmark networks explored include the two-loop network, Hanoi network, the New York City water supply tunnels, the Anytown water distribution system, the two-Reservoir Gessler pipe network and Selebi-Phikwe network.

Chapter 4: Study Area, Methodology and Data Collection.

This chapter begins by describing the study area and highlighting the information required for the design of a water distribution network gathered from project documents from SFCE Consulting Engineers and Mogale City Local Municipality. The methodology employed in assessing the hydraulic performance of the existing network and the design of the upgraded network is also explained in this chapter. The hydraulic performance of the existing WDN is undertaken and finally, the proposed upgraded network is then optimized using trial and error and Wadiso software.

Chapter 5: Results and Analysis of Findings. The results of the hydraulic performance of the existing WDN, the optimization of the upgraded WDN by trial and error, and Wadiso are presented and discussed in this chapter.

Chapter 6: Conclusion

The report is concluded by summarising the research undertaken to address the objectives of the study and suggesting areas future research.

2. LITERATURE REVIEW

2.1 Water Demand

Adequate and safe water supply is one of the basic services which are highly influencing the economic progress and the health of communities. Since water resources availability is related to economic and social development, it implies that development is influenced by water resources availability and management. The design of water distribution and storage infrastructure requires a robust estimate of water demand (DHS 2019). There are a lot of factors that influence the water demand of a particular area such as topography, sanitation system, development of the community and water demand management in a particular community. There is no consensus on what is understood by the term demand, as engineers, social scientists and economists present different points of view. Water demand therefore includes the water supplied to the entire community to meet the needs of the intended recipients, water supply for firefighting, system flushing, water required to properly operate treatment facilities and leakages in the system. It is important to understand the nature of demand to avoid wasting resources or constructing unwanted or inappropriate systems.

Water demand varies seasonally, normally with lowest demand during the winter months when it is cold and the highest during summer months when it is hot. Variations in water demand also occur with respect to time of the day. Diurnal peaks normally occur in the morning and early evening, while the lowest demand occurs at night when people are sleeping. Generally, peak hours occur when water is used for cleaning, food preparation, ablution and washing. The flow capacity of a WDN should be adequate to cater for the network's instantaneous peak water demand and this is obtained by multiplying the summer daily demand value by a daily peak factor. Normally, the storage stores water during the night so that it can be supplied during the peak demand. For efficient water distribution, it is required that water should reach end users at the required pressure with required flow in the network.

A WDS is designed to cater for domestic and non-domestic water requirements. Non-domestic water requirements include water use for schools, hospitals, clinics, bus stations, community halls, restaurants and for stock (CSIR 2000). The main factors that influence water demand are development level and the type of sanitation in that area (CSIR 2000). Areas with very high development levels include formal

suburbs which are fully reticulated whilst those with high development levels include extensive formal suburban housing with flush toilets. In contrast moderate development levels comprises medium sized formal housing with limited finishings. Developed areas normally have services installed at the highest level and will not require future upgrades but developing areas may be subject to future upgrades to a higher level.

2.2 Water Distribution Network

A Water Distribution Network (WDN) is a system that distributes water to end users and comprises pipes, reservoirs, pumps, and valves of different types. WDNs are vital urban infrastructure and require significant investment (Abebe and Solomatine 1998). Significant research has been done over the years to assess the behaviour and performance of WDNs using hydraulic modelling software such as EPANET (Mala-Jetmarova *et al.* 2018).

One of the primary objectives of a water supply utility is for its WDN to operate efficiently at the least cost throughout its design life. To achieve this objective, it is imperative to evaluate the efficiencies of existing components of the WDN using suitable performance indicators (Haider *et al.* 2014). The performance of a WDN can be defined as its ability to deliver a required quantity of water under sufficient pressure and at an acceptable level of quality during different normal and abnormal operational situations (Tabesh and Doulakhah 2006). Performance assessment is defined as, any approach that allows for the evaluation of the efficiency or the effectiveness of a process or activity through the production of performance measures (Muranho *et al.* 2014). Since water distribution systems (WDS) involve substantial investment from the owners of the asset and are not easy to design and operate, their designs must be well thought through.

The design of WDNs involves sizing and locating various components of the network. These components determine the cost of the WDN and the level of service to be provided to end users. WDNs are designed to meet future peak demands as required by the consumers. The optimum design of a water distribution system is crucial to supply water with adequate pressure and flow. If the system is not efficiently designed, problems might occur during operation leading to poor performance and/or increased operating costs over the system's service life and this

is undesirable. Optimal design consists of choosing a diameter for each of the pipes in the system and thereby ensuring that the pressure in each one of the nodes is greater than or equal to a minimal allowable limit (Páeza *et al.* 2014). The analysis and design of pipe networks is not an easy task especially when the network consists of a range of pipes, as is typical in big urban centres. Ramana *et al.* (2016b) highlighted that the computation of flows and pressures in a complex network, is a challenge and interest for designers, contractors and those who maintain public water distribution systems. On the other hand, modelling rural networks is often challenging task due to very low pressures in networks due to lack of maintenance or leakages coupled with restricted water supply for certain hours in a day due to low water supply (Ramana *et al.* 2016a). Liong and Atiquzzaman (2004) also indicated that designing an optimal water distribution network is a challenging undertaking.

The history of design, operation planning of WDN's dates to the late 19th Century (Mala-Jetmarova *et al.* 2015). There are various methods that have been used to compute flows in WDN's (AWWA 2005, as cited in Ramana *et al.* 2015) which include graphical methods, physical analogies, and mathematical models. These methods have been developed, implemented, or partially implemented over time. The Hardy Cross Method is one of the earliest theories developed to determine water flow and pressure in WDN's (Ramana *et al.* 2016). It is an iterative method used for the determination of the flow in the pipe and pressures at nodes where the flow inputs and outputs at nodes are known.

The first step in the design of a WDN is establishing the design criterion. The traditional design criteria used are based on water quantity, storage, service pressure, and flow (Briere 2014). The design criteria are normally enshrined in local authority or national guidelines. Most local authorities in South Africa, traditionally use the Red Book (Guidelines for Human Settlement Planning and Design 2000), which was recently updated to DHS Red Book (The Neighbourhood Planning and Design Guide 2019), to supplement guidelines developed by national and provincial government departments, statutory bodies and local authorities (DHS 2019). Major cities such as City of Tshwane, City of Johannesburg, City of Cape Town and other metropolitan centres also have their own local guidelines.

Water quantity is generally estimated with a provision for anticipated future needs considering expected population growth. It is desirable that the water supply be large

enough to meet different conditions. Furthermore, engineers are legally required to design a WDN which meets the maximum daily demand plus the fire flow demand (Briere 2014; Ilemobade and Stephenson 2006).

Generally, the design of water distribution networks comprises the following steps:

i. Determination of the water system's design period.
ii. Estimation of the demands at the nodes.
iii. Determination of design criteria as per National, Provincial, District or Local guidelines and standards.
iv. Establishing (or fine-tuning) the network configuration.
v. Selecting the pipe diameters to be used and material type.
vi. Developing the model and analysing it to determine the hydraulic performance of the proposed network solution.
vii. Optimization of the network by trial and error, optimization algorithms or software such as Wadiso.
viii. Repeating steps (iv) to (vii) until a satisfactory solution is found.

The design process therefore encompasses a variety of considerations, from WDN model development and calibration, through to the selection of network elements as the system evolves over time, to operational issues such as reliability, energy consumption, water quality deterioration, water quality and quantity (Savić *et al.* 2018). Due to the complex nature of WDNs, their design and analysis is normally undertaken using software such as EPANET, which is widely applied globally, both for its hydraulic and quality analysis of WDNs.

2.3 EPANET

EPANET is a public domain software developed by the US Environmental Protection Agency (EPA) and is widely used to design and analyse the technical sustainability of water supply networks (Hossain *et al.* 2021). EPANET performs extended period simulation of hydraulic and water quality behaviour within pressurized pipe networks. Gómez *et al.* (2016) describes EPANET as the benchmark software for design and analysis of pressurised water distribution networks which is both reliable and user friendly. EPANET has been chosen for this study because it is meant for public and educational use and is free available online (Ramana *et al.* 2016b; Liong and Atiquzzaman 2004). This computational tool allows for solving of hydraulic equations

based on the conservation of mass and energy through the use of systems of linear and nonlinear equations, such as the Newton-Raphson technique, based on the use of a Demand Driven Analysis (DDA) (Rossman *et al.* 2020). According to Shital *et al.* (2016) the simulation of existing WDNs using EPANET helps water supply engineers save time as the use of EPANET is fast and less tedious than hand calculations. EPANET makes it easy for designers to read and interpret results because it uses various data reporting visualisations such as colour-coded network maps, time series graphs, contour plots and data tables.

By using EPANET for the assessment of hydraulic performance, 'various hydraulic parameters, such as base demand, pressure head, head of water and friction factor can be easily observed even for large or complex networks (Ramana *et al.* 2016). During the analysis, pipes can be set open or closed at pre-set times or when specific conditions exist, such as when tank levels fluctuate or when nodal pressures fall below or above certain values (Rossman *et al.* 2000). Designers utilize EPANET to design new infrastructure, investigate water quality problems, lower energy consumption, retrofit existing old infrastructure, optimize operations of tanks and pumps, and prepare for emergencies (Rossman 2000).

2.4 Skeletonization of the network

Skeletonization of a WDN entails combining a series of pipes with one single pipe to reduce its size but with minimal impact on the system behaviour. Skeletonized network models typically include only those pipes that are considered major to the flow and distribution of water. For example, a skeletonized model might only consider 100mm and larger diameter mains, which are significant to the flow and distribution of water for steady state simulation. This process makes it easier to define the network since there is less data involved, and the simulation time is shorter because other pipes will not be involved in the formulation of a solution. Some of the advantages of skeletonization include that the displaying of results is quicker and more readable since less pipes are involved and that the number of iterations required to converge to a solution is typically less as well.

The procedure has its own limitations, and it is recommended not to be used when performing water quality modelling since the flow rates, paths, and velocities for all pipes are critical components of a water quality simulation. However, for steady state water distribution simulation, skeletonization can only produce results that have

sufficient accuracy if the water consumption has been properly assigned to the defined nodes.

2.5 Hydraulic Simulation

The simulation of a WDN is a process of imitating its behaviour using a mathematical representation of real system (Hamed 2020). Furthermore, WDN simulation is used for network design and evaluation, long term planning of new developments, rehabilitation studies, fire protection studies, water quality studies and energy management (Hamed 2020). The utilization of various software in simulation of WDNs provides and improves the replication of nonlinear dynamics of WDN by solving steady state hydraulic equations that include conservation of energy and conservation of mass elements (Ayad *et al.* 2013; as cited in Ramana *et al.* 2016). The simulation can either be Steady State (SS) simulation or Extended Period Simulation (EPS).

2.5.1 Steady State Simulation

The steady-state behaviour of a WDN is governed by the laws of conservation of mass at the nodes and conservation of energy around the loops of the network as well as the nonlinear head loss/ flow relationship in the pipes. This involves the calculation of flowrates and pressures for a steady state pipe network system WDN which represents a snapshot of WDN under certain conditions. This analysis computes the pipe flowrates and the node hydraulic grade line (HGL) elevations so that the conservation of energy and mass are satisfied. These steady state conditions reduce to the following formulae below.

The principle of conservation of mass states that the fluid mass entering a node will be equal to the mass leaving that node. This can be represented with equation (2.1) below:

$$\sum(Q_{in} - Q_{out}) = 0 \qquad (2.1)$$

Where, Q_{in} = *flow into node*

Q_{out} = *flow away from node*

The principle of conservation of energy states that the difference in energy between two points in a loop must be the same regardless of the path taken.

$$\sum h_f = \Delta E_{FGN} \qquad (2.2)$$

Where, $\sum h_f$ = *head loss between two points in a loop*

ΔE_{FGN} = *difference in total hydraulic grade between fixed grade nodes (FGNs).*

The head loss relationships states that the algebraic sum of the head loss in a loop must be equal to zero (Swamee and Sharma 2008)

$$h_f = g(Q_i) \quad (2.3)$$

Where, $g(Q_i)$ = *pipe head loss equation as a function of flow,* Q_i.

Reducing these formulae yields nonlinear algebraic equations which cannot be solved directly but by methods such as the Hardy Cross, Newton-Raphson, and linear theory. Ilemobade and Stephenson (2003) states that efficient hydraulic simulation is based on modelling the WRN using the above equations (i.e., equations 2.1 to 2.3) and determining the flow in the pipes and residual pressure heads at the nodes.

Hardy Cross Method

The Hardy Cross method was the first attempt to solve a WDN based on systematic iterative procedure for network analysis (Cross 1936, as cited in Hamed 2020). The method employs the principles of conservation of mass and conservation of energy, and it is an iterative method for determining the flow in the pipes where inputs and outputs are known (Ramana *et al.* 2015). It is an adaptation of the Moment distribution method, developed by Hardy Cross for the determination of moments in indeterminate structures (Ramana *et al.* 2015). This method is based on the basic equations of continuity of flow (equation 2.1) and head loss (equation 2.3).

2.5.2 Extended Period (Dynamic) Simulation

Extended Period Simulation is used to analyse a pipe network for an extended period which represents the hydraulic performance of a WDN over a given time span, usually 24 hours (Hamed 2020). The hydraulic time step for an extended time simulation is usually set by the user and the default value is one hour (Rossman *et al.* 2000). At each time step an analysis is conducted for the pipe network based on the current network parameters and the pipe flow rates calculated from the previous time step.

EPANET's hydraulic simulation model calculates hydraulic heads at junctions and flow rates through pipes for pre-set reservoir levels, tank levels and water demand over a succession of points in time (Rossman *et al.* 2020). These parameters are updated from one time step to another as per prescribed time patterns, while tank levels are updated using the current flow solution. The solution for heads and flows at a particular point in time involves solving the conservation of flow at each junction and the head loss relationship across each link in the network, in a process which is known as hydraulic balancing which uses an iterative technique to solve the nonlinear equations involved.

2.5.3 Demand Driven Analysis

Conventional methods for analysing WDNs are demand-driven i.e., nodal demands are fixed, and pipe flow rates are adjusted so that demands are met in full. The implied assumption is that sufficient pressure will be available (Ackley *et al*. 2001). The demand driven analysis (DDA) is formulated assuming that demands at nodes will be satisfied regardless of the pressure, and it yields lower or even negative nodal pressures while analysing a pressure deficient network. The DDA is regarded as the standard approach in hydraulic modelling of WDNs in South Africa with fixed demands at each node (Jacobs and Strijdom 2009).

The aim of DDA is to determine pressure heads at the demand nodes and to ascertain that the pressure heads are greater than or equal to the specified minimum head. If not, the network is considered deficient and negative pressure problems may arise in some nodes. Therefore, DDA helps to locate negative pressure nodes (deficient nodes). To rectify this, some modifications include increasing heads at source nodes, adding one or more booster pumps in the network and providing check valves.

Vaidya and Mali (2019) highlighted that in this approach, nodal pressure head and pipe flow are computed from the equation of continuity and energy by assuming fixed nodal demands. Thus, it assumes that available pressure at each node is above the minimum required pressure under normal conditions. This assumption makes the mathematical solution easier even though it might not be appropriate because the amount of outflow at nodal outlets depends on network pressures.

Hamed (2020) argued that the main drawback of the DDA is its inability to assess the network in deficient conditions, which may produce wrong behaviour in the elements of the WDN. Chang *et al.* (2021) added that DDA has limitations in hydraulic analysis for abnormal conditions due to the assumption that the base demand of nodes is always satisfied regardless of the node pressure condition. Under these conditions, the traditional DDA will compute heads that are below the minimum required for outflow to occur physically at some or all the nodes (Ang and Jowitt 2006).

Ackley *et al.* (2001), concluded that DDA is inherently unsuitable for use in the abnormal circumstances of a WDS under stress. This has been corroborated by other researchers including Ozger and Mays (2003) and Hamed (2020). It can therefore be concluded that this approach is preferred when there are no pressure deficient conditions in a water supply system because it fails to represent the real scenario when a pressure deficiency problem happens in the network.

2.5.4 Pressure Driven Analysis

As previously stated, under normal operating conditions, the associated pressure regime proves to be adequate and can be analysed by DDA. However, when the network is under stress from abnormal operating conditions, such as exceptionally high demands or failure of network components and low pressures at some nodes, DDA cannot represent this situation (Ackley *et al.* 2001). Recent research has shown that the Pressure Driven Analysis (PDA) generates much more realistic results under partially failed conditions of a network (Ozger and Mays 2003). The PDA assumes that the demands on the nodes rely on pressure. Therefore, the PDA provides a more realistic approach and is recommended for use primarily for reliability assessment of WDNs.

Siew and Tanyimboh (2012) highlighted that pressure deficient conditions are inevitable in WDSs and can be caused by common occurrences such as pump failure, pipe bursts, isolation of major pipes from the system for planned maintenance work and firefighting demands. Hamed (2020) also highlighted that partially failed conditions within a network may result from abnormal demands from clients, power outages, valve failure, and even terrorist attacks.

PDA utilizes nodal head-discharge relationship for analysing pressure deficient conditions. This has been supported by Vaidya and Mali (2019), and Hamed (2020). The available nodal flow for PDA can be obtained by various methods but they are generally categorised into two. According to Siew and Tanyimboh (2012), the first category comprises methods involving DDA, and the second category involves the approach where a head-flow relationship (HFR) is embedded in the system of hydraulic equations. HFRs are functions used to estimate the actual flow at demand nodes based on the nodal pressure (Siew and Tanyimboh 2012).

The relationship between the available head and required head at a node is shown in equation (2.4) below.

$$H^{avl} = H^{req} \quad (2.4)$$

Where, H^{avl} is the available head at a node and H^{req} is the required head at a node. According to Vaidya and Mali (2019), in case of abnormal conditions, the amount of water at a node is dependent on the available pressure and the relationship between the flow and pressure at a demand node is called node head-flow relationship (NHFR). The NHFR has been represented using different equations by different researchers including Bhave (1981), Geramnopoulos (1985), Wagner *et al*. (1988b), Kovalenko *et al*. (2014) and others (Vaidya and Mali 2019). For Bhave (1981), the NHFR was depicted as shown below:

$$q_j^{avl} = q_j^{re} \text{ (adequate flow), if } H_j^{avl} > H_j^{min} \quad (2.5)$$

$0 \leq q_j^{avl} \leq q_j^{req}$ (no flow, partial flow or adequate flow) if $H_j^{avl} = H_j^{min}$

$$q_j^{avl} = 0 \text{ (no flow) if } H_j^{avl} < H_j^{min} \quad (2.6)$$

PDA of WDNs can now be performed using the latest version of EPANET, version 2.2. EPANET 2.2 provides an option to select a demand model to use when setting the analysis options and both the DDA and PDA are available for selection.

2.6 Network Configurations and Methods

Pipe layouts normally follow road layouts because they are usually laid below road pavements. This is the easiest arrangement because most often, road assets and pipe networks are owned by the same authority and thus, eliminates the laborious process of land negotiations. There are, in general, four pipe network layouts and

these are grid iron, ring, radial and dead-end system as depicted in Figure 2.1 to Figure 2.4.

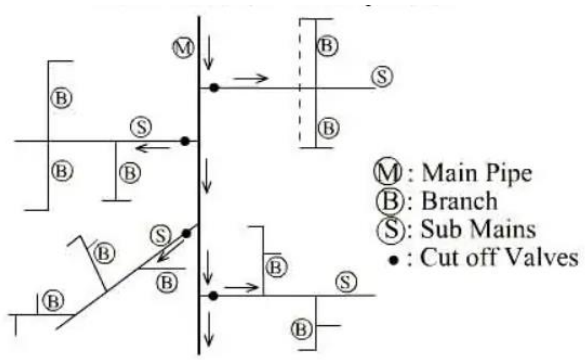


Figure 2.1 : Dead end system

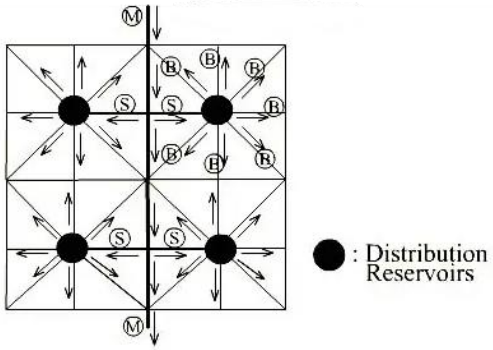


Figure 2.2 : Radial System

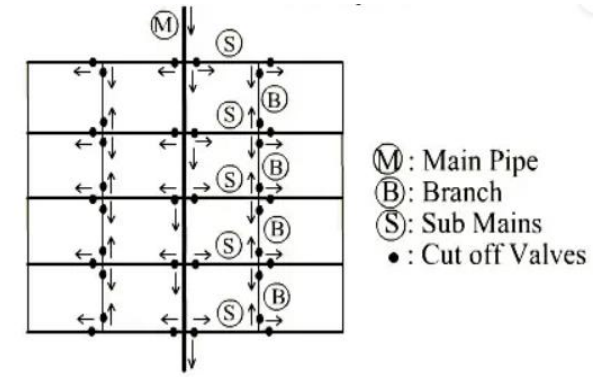


Figure 2.3 : Grid iron system

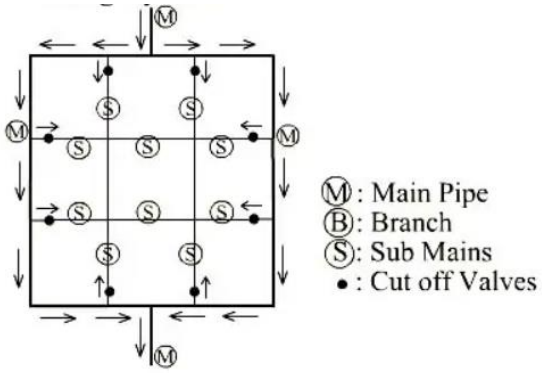


Figure 2.4 : Ring system

The choice of network layout is influenced by the number and type of service connections. Service connections comprise house, yard, and communal standpipes.

According to Trifunovic (2006), WDSs can be differentiated either by the type of distribution scheme or network configuration. The WDSs can be distinguished with respect to the way the water is supplied. There are three types of distribution schemes which include gravity, direct pumping and combined. The existing topographical conditions normally influence the choice of the distribution scheme. In a gravity flow reticulation system, the reservoirs or tanks are located at a higher point than all other areas of the WDN and there is no need for pumping (DHS 2019) whilst for a pressurised system, water flow is aided and controlled by pumps.

2.7 Trial and Error Optimization

The design of a WDNs involves sizing the elements of that network (e.g., pipes) whilst trying to strike a balance with other conditions such as cost, reliability etc. The determination of optimum pipe sizes is normally a complex exercise (Mala-

Jetmarova *et al.* 2018; Boindala and Ostfeld 2022) that requires understanding and implementation of optimization methods. However, some designers rely on trial and error to determine the most efficient design. To achieve this, these designers adjust pipe sizes using engineering judgement to satisfy the design criteria and reduce costs (Ilemobade and Stephenson 2006). Experienced designers may use rules of the thumb, such as head loss per unit length, to eliminate impractical combinations during the trial and error process. Geem (2006) highlighted that, to find low cost designs, some engineers use trial and error based on their intuitive engineering sense, but this does not guarantee optimal or near optimal designs. This was confirmed to be a common practice amongst water systems designers when Ilemobade and Stephenson (2006) compared the results obtained by trial and error with those generated using the nonlinear optimization method employed in Wadessy for the Selebi-Phikwe water reticulation network. There was 62% reduction in total costs achieved when Wadessy was employed in comparison with trial and error. When using trial and error, several network configurations and designs are identified and investigated in relation to the design criteria. This becomes difficult even for experienced designers when dealing with a WDN with many pipes because the method becomes tedious, and time is wasted trying to find an optimum design. Therefore, the desire to provide better tools that can assist in determining efficient designs resulted in the formulation of several optimization approaches in the 1980s (Mala-Jetmarova *et al.* 2018, Geem 2006).

Several optimization studies (such as Ramana *et al.* 2016b, Shital *et al.* 2016) which used EPANET for simulation of WDNs relied on trial and error or engineering judgement to determine pipe sizes. Ramana *et al.* (2016b) indicated that having known the residual pressures at all nodes and the corresponding flows in pipes, the diameter of the pipes could be fixed by trial and error. Therefore, it is not surprising for designers to use trial and error coupled with engineering experience. The result of assigning pipe diameters in this way is that the design criteria are met without adequately considering the cost implications because a manual iterative (trial and error) approach is not effective where the number of combinations is large.

From the continuity equation ($A_1V_1 = A_2V_2$,), an increase in pipe diameter would correspond to a decrease in velocity in the pipe. Likewise, the challenge with trial and error is that selecting a larger pipe diameter can reduce head loss and increase

pressure at junctions, but the use of larger pipe diameter means increased pipe cost. On the other hand, using smaller diameter pipes will reduce costs but might result in insufficient node pressure challenges due to increased head losses.

2.8 Optimization

Traditionally, the optimization of a WDN design involved determining sizes, locations, and operational statuses of network components (such as pipes, pumps, tanks, and valves) while keeping the system capital and operational costs at a minimum. This involves minimizing initial construction costs and the present value of operating the WDN. Ultimately, the optimal cost design is the one with the least total cost out of several designs. In many WDS designs in Southern Africa, optimization is normally not undertaken, with the pipe sizing done by trial and error and this renders the WDS under-designed (or over-designed) and prone to failure. Ilemobade and Stephenson (2006) highlighted that most systems are under designed due to, amongst other things, lack of tools and skills. Therefore, while WDN optimization algorithms by researchers have been extensive, they are less popular in practice. With regards to gravity WDNs, a basic optimization model will seek to reduce the design cost of the system subject to meeting the minimum pressure at individual nodes and with pipe diameter or size as the only decision variables (Mala-Jetmarova *et al.* 2018). This ensures that a cost effective WDN is designed and that the WDN meets the required water demands, at desired pressure heads. The optimization of pumped WDNs is complex because of the presence of pumps and tanks, which have their own requirements that need selection or variation such as location, sizes, and operational statuses.

The optimization problem can be described by the objective function, constraints and decision variables. The objective function, which is to be minimized, shows the total cost including the present value of operating costs with constraints which are that the pressure at selected nodes is to be within a specified range and demands are to be met Alperovits and Shamir (1977). The decision variables include design parameters (pipe diameters, pump capacities and reservoir elevations), and operational parameters (pumps and valves settings).

In a single objective optimization procedure, there is only one objective function, for example the cost of the pipe, with the pipe diameter regarded as the decision

variable (Shende and Chau 2019). As per Geem (2006) the least-cost design can be stated as follows:

Minimize: Cost of the water network design, subject to:

i. Continuity
ii. Conservation of energy equation
iii. Minimum pressure requirements
iv. Other constraints (maximum pressure; flow velocity)

The objective function (total cost of the WDN) can be expressed as:

$$Minimize\ total\ cost = \sum_{i=1}^{n} L_i \text{ x } c(D_i) \qquad (2.7)$$

Where n = total number of pipes and $c(D_i)$ = cost per unit length of the ith pipe in the distribution system of length L. The continuity equation and conservation of energy equation are given in section 2.5.1 and the minimum pressure requirement according to Geem (2006) is given in the following form:

$$H_j \geq H_j^{min}\ ; \qquad j = 1, \ldots, M \qquad (2.8)$$

Where, H_j = pressure head at node j,

H_j^{min} = minimum required pressure head at node j, and

M = number of nodes in the network.

Several researchers have presented different methods of optimization for obtaining the optimal WDN in a bid to minimize costs whilst considering demand satisfaction (Mala-Jetmarova *et al.* 2018). These optimization methods include linear programming, nonlinear programming, genetic algorithms, simulated annealing, tabu search and the harmony search.

Researchers have faced several challenges associated with traditional optimization approaches such as difficulties associated with formulation ease, fast convergence, handling non-linearity and handling discrete diameters (Shende and Chau 2019), and this has led researchers to use these algorithms with some deterministic mathematical-based approaches. Other researchers concluded that it is complicated to achieve an optimal network design because of the nonlinear relationship between flow and head loss and the availability of pipes on the market in discrete sizes (Liong

and Atiquzzaman 2004). These challenges coupled with the nature of the objective functions such as the cost, which is nonlinear, make the optimization of networks complex.

As discussed, WDS design problems were traditionally formulated as single-objective optimization (i.e., least cost) problems. The challenge with this approach was that even though it sought to find the best solution for cost, there was a possibility that it could result in low performance during adverse conditions such as power outages, pipe bursts or pipe aging. Therefore, a multi-objective optimization approach was introduced to maximise the network benefit (e.g., reliability, redundance) on the one hand and minimise system cost on the other hand. A multi-objective optimization approach is considered a good approach by engineers as it offers a tool to explore interesting trade-offs, for instance, a marginal pressure deficit can be outweighed by a significant cost reduction (di Pierro *et al.* 2009). By formulation, multi-objective design is considered a large-scale optimization problem, which is both nonlinear and discrete. Multi-objective optimization problems including two objectives, three objectives and even six objectives have been extensively explored in the WDS sector (Boindala and Ostfeld 2022).

According to Mala-Jetmarova *et al.* (2018), the single-objective method has the advantage of finding one best solution, which is easy to analyse and execute. In contrast, the multi-objective methods yield a set of trade-off solutions, which require choosing one or a limited number of promising solutions. Some studies consider a single economic objective to formulate a network rehabilitation problem, while succeeding investigations utilize a multi-objective optimization framework so that they can incorporate measures affecting the level of service provided to customers (Mala-Jetmarova *et al*. 2018).

WDS optimization problems can be broadly grouped into design (or component sizing) and operational problems. The design (component sizing) problems include sizing of pipes, tanks, pumps, and valves. In contrast the optimization of WDS operations includes constraints of pump operation, water quality assessment and valve control. Similar to the WDS design optimization, the optimization of WDS operations has been studied by various scholars and different solutions have been proffered (Savić *et al.* 2018). A third group of optimization problems is related to WDS model calibration, reliability, and system partitioning.

Optimization problems are solved using various optimization techniques that date back to the beginning of the 20th century before the advent of the digital era. Since the 1970s, there have been several studies on optimization of WDSs (Mala-Jetmarova *et al.* 2015). Optimization methods are categorised into deterministic and Metaheuristic methods.

2.8.1 Deterministic optimization methods

Deterministic optimization methods include linear programming, dynamic programming, and nonlinear programming. They have been found to utilize the advantage of the analytical model of the problem to produce a series of candidate solutions, which then aim to converge to an optimal solution (Savić *et al.* 2018). This has been highlighted as the main attribute of the linear programming technique.

Linear Programming (LP).

Amongst deterministic optimization techniques, the linear programming ranks as one of the most widely employed techniques of deterministic optimization (Awe *et al.* 2019, Savić *et al*. 2018). The use of LP in WDS requires linearization because WDS optimization problems are nonlinear since the relationship between flow and head loss is nearly quadratic. Awe *et al.* (2019) further highlights that the linear programming technique guarantees an optimal solution for a continuous problem with a linear objective function subject to linear constraints.

Dynamic Programming (DP).

For multistage optimization problems, the dynamic programming method is utilized. Its approach involves decomposing the multistage problem into a series of single-stage decision operations (Savić, *et al*. 2018). Some researchers found that this technique is more appropriate in the scheduling of pumps (Ilemobade *et al.* 2005). Its disadvantage was presented as a problem of dimensionality and its limited application to complex WDSs (Awe *et al.* 2019).

Nonlinear Programming (NLP).

The third technique amongst deterministic methods is non-linear programming techniques, which are based on generalised methods of minimized gradient, sequential linear programming, or sequential quadratic programming (Zhang *et al.* 2019, as cited in Awe *et al.* 2019). The similarity between the linear and non-linear programming techniques is that they both use continuous variables but then differ on computation of the solution. When used on a WDS, the non-linear programming

does not mandatorily converge to the optimal solution (Awe *et al.* 2019). Furthermore, Awe *et al.* (2019) highlights that the limitation of non-linear programming is that it can only handle WDSs of low complexity because of the number of constraints and variables it can manage.

2.8.2 Metaheuristics methods

Metaheuristics methods emerged in the mid-1990s (Savić *et al.* 2018), and they comprise of general-purpose algorithm designed to solve a wide range of difficult optimization problems without having to be adapted to each problem. Awe *et al.* (2019) defines metaheuristics or stochastic methods as strategies that have been formulated to produce partial search algorithms, which may then supply the best solution with imperfect or incomplete data. They further describe these methods as high-level algorithms which were devised to solve complex optimization problems.

The metaheuristics algorithms share some common characteristics such as, they are stochastic in nature, they are nature inspired, they do not require linearizing assumptions like some deterministic techniques, and they normally have several parameters that need to be tuned for each problem to improve performance (Savić *et al.* 2018; Nicklow *et al.* 2010 as cited in Awe *et al.* 2019). Metaheuristic methods have been found to be better than deterministic optimization approaches when applied to large and complex problems which cannot be solved by deterministic algorithms. Mala-Jetmarova *et al.* (2015) highlighted various metaheuristic methods that have been applied to WDS optimization such as genetic algorithms and their numerous variants, simulated annealing, shuffled frog leaping algorithm, shuffled complex evolution, harmony search, cuckoo-search algorithm, particle swarm optimization, scatter search, immune algorithm, memetic algorithm, honey bee mating optimization, differential evolution, mine blast algorithm, and discrete state transition algorithm.

Optimization tools have been found to be efficient in saving design time and the associated costs, especially in large networks (Ilemobade and Stephenson 2006). They are also able to produce more feasible solutions, thereby making the determination of an optimal solution simpler. Mohan and Babu (2009) concluded that in recent years, stochastic optimization algorithms like genetic algorithms, colony optimization and simulated annealing have been found to be successful in determining the optimum pipe size that satisfies the head requirement at the

minimum cost. It is therefore imperative to utilize an optimization technique to achieve an optimum design rather than base the designs on trial and error.

The other optimization method that falls under this category of metaheuristics methods is the partial enumeration technique, which evaluates all alternative solutions to obtain the optimum solution. The partial enumeration technique follows a series of steps to obtain the solution such as those presented in the flowchart in Figure 2.5 (GLS 2023).

2.9 Wadiso

As alluded to in the preceding sections, the Wadiso 6 software was employed to optimize total pipe costs in this study. The Wadiso 6 Quick Start Guide (GLS 2023) defines the Wadiso program as a robust analysis and planning software for WDSs. The programme operates in an environment built in CAD and GIS, called Albion. It includes functions which can capture data, edit models, and view and interpret results (GLS 2023). It has capabilities to perform steady state analysis, optimization, extended time simulation and water quality simulation of complex WDSs. The Wadiso software runs on an EPANET engine with an optimizer solver added for optimization.

The Wadiso software produced by the GLS Software company in South Africa is based on the optimization techniques developed by Prof. Johannes Gessler at Colorado State University, Fort Collins, Colorado (USA) in the 1970's GLS (2023). It was initially formulated as a basic numeric algorithm to solve a set of head loss and continuity equations. All the other capabilities, including optimization, were added later. The program was then improved i.e., user friendliness, optimization, speed and interfacing with other software such as CAD by GLS (2023).

The optimization module of Wadiso computes the least cost combination of pipes, pump and tanks that meet the specified design constraints, such as load cases (demand) and minimum required pressures. It uses a simple algorithm that is based on exhaustive enumeration. Initially the designer is required to specify the sizes for each pipe and the algorithm then tests all possible combinations of pipe sizes and checks if they comply to pressure requirements. The exhaustive enumeration allows for the establishment of a queue of Pareto Optimum solutions, including solutions that may be more expensive than the optimum solution but providing better pressure

or solutions that slightly do not conform to the pressure requirement but are cheaper (Loubser and Gessler 1990). A combination of pipe sizes is defined as a Pareto Optimal solution when there is no other solution with better pressure at a lower cost. This is achieved by relaxing the minimum pressure constraint and allowing solutions which result in costs within a certain range from the minimum cost. These solutions are sub-optimal solutions as they give higher pressures at an increased cost, or which slightly violate the pressure requirements at lower cost.

2.9.1 Exhaustive Enumeration

This exhaustive enumeration method seeks to minimize the cost of the installation and operation of the system while satisfying demand and hydraulic criteria just like any other optimization method. The approach involves specifying a range of candidate diameters for all pipes to be sized, then the enumeration algorithm generates all possible combinations of pipe diameters and test each combination to check if it meets the prescribed pressure constraints. The least expensive combination that does not violate the pressure constraints is the preferred combination (Loubser and Gessler 1990).

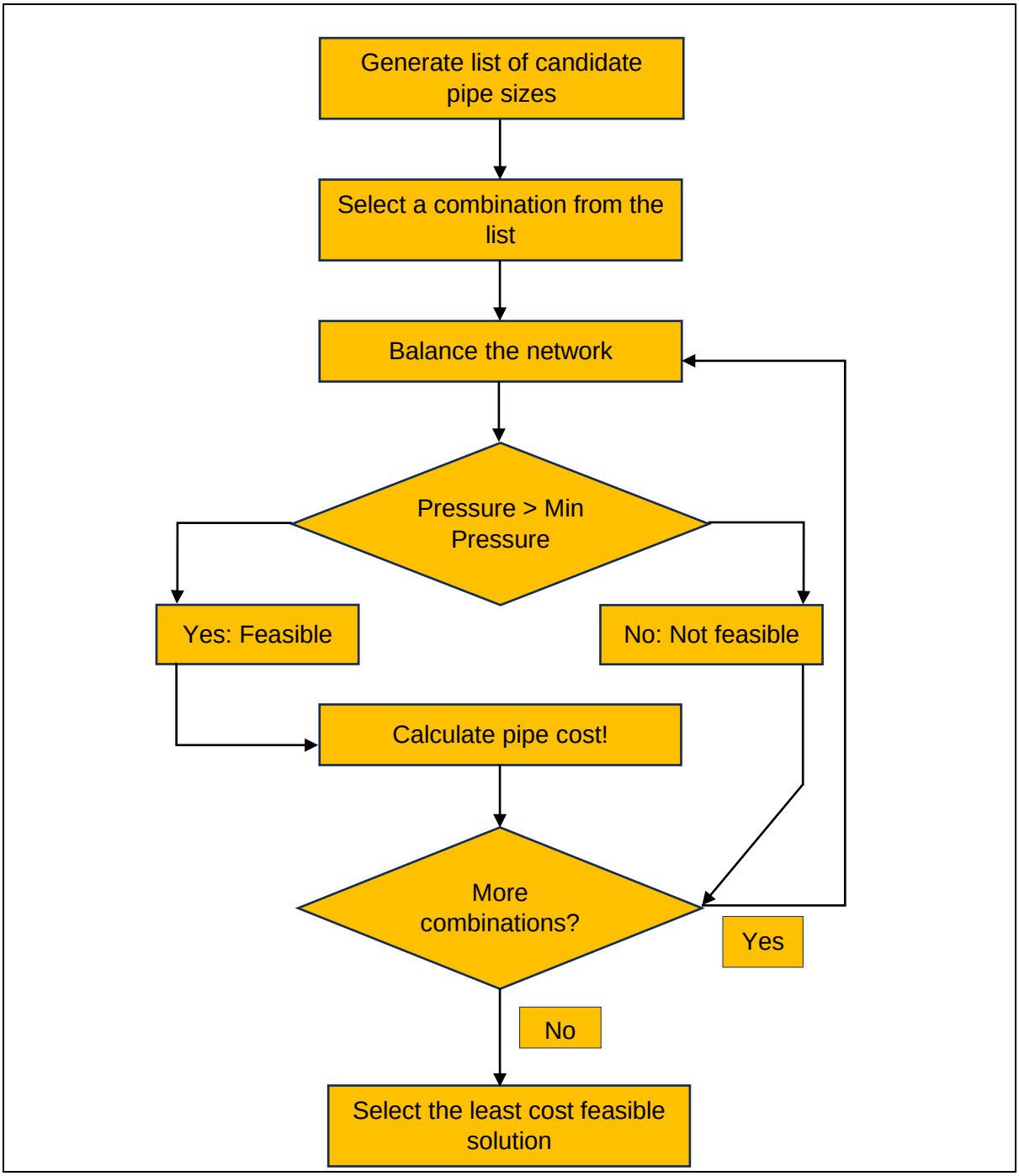


Figure 2.5 : Enumeration Algorithm Flowchart (GLS 2023)

With this method, large systems often present a challenge of longer periods of computation. To reduce the enumeration technique computation time, Gessler (1985) developed the partial enumeration technique.

In the partial enumeration technique, the computation time is minimised by grouping of pipes, testing on size range, cost testing and size testing. The pressure distribution is computed only after pipe size combination has been subjected to all the four techniques listed. It is an exhaustive method that can be employed to

determine the best solution for optimal pipe diameters for a given network. The global optimum solution is realised after all the alternatives are tested during the optimization process. The intent of the Wadiso optimization routine is to use these techniques to size specific pipes in the system and not all of them thereby reducing the computation time. The four techniques used in partial enumeration described below:

i. Grouping of pipes

This technique involves combining pipes in the model into a single group based on their location in the network. A group consists of one or several pipes to be assigned to the same diameter, during optimization. Pipes from same proximity might be treated as single pipe (group in Wadiso) instead of different pipes which reduces the number of combinations of solutions. Gessler and Walski (1985) argue that pipe diameters do not change at every block in a distribution system and hence it is prudent to group pipe within the same proximity as a single size pipe. Where no other pipes sizes are within the range of one pipe size, then it is also possible to treat that one pipe size as a group.

ii. Test on size range

This follows an iterative route whereby it is established whether the smallest pipe size specified for each group can meet the pressure required when combined with the maximum pipe sizes in all other groups. If the proposed smallest size in a group fails the test, that size in the group is eliminated and the second smallest size is tested.

iii. Cost test

Like the other three techniques employed in partial enumeration, the objective of the cost test is to reduce the number of candidate solutions. The testing of combinations in partial enumeration is terminated when a size combination is found that meets all pressure requirements. This works on the basis that there is no need to further compute the pressure distribution for any other system that is more expensive than the least expensive combination. Each pipe to be sized is assigned to a cost function and pipes in the same group may even be assigned to different cost functions. A cost function represents a list of pipe sizes, and, for each size, an installation cost (GLS 2023). Different cost functions may be

specified for the same pipe size depending on where or how the pipe is installed. For example, a pipe installed in a developed central business district will cost more than a pipe of the same size installed in a new subdivision.

iv. Size test

The size test is achieved by testing pipe size combinations which would have passed the cost test, to check if they meet the pressure requirements. If a certain combination of pipe sizes does not meet the pressure requirements, then no pipe size combination will meet the pressure combination. Sizes refer to a list of discrete pipe sizes from which the program selects the optimum size for a particular group (GLS 2023).

2.10 Wadessy

The reduction of total pipe costs by utilising Wadessy (Ilemobade and Stephenson 2003) for optimization of the case study network was initially investigated in this study but running Wadessy's optimization was not possible because of compatibility issues with the Windows 10 and Windows 11 Operating Systems. The optimization was then performed with Wadiso. The attributes of Wadessy are however still discussed in this section.

With the aim to be used in Southern Africa, Wadessy is a valuable tool for the design of optimum WDNs. Wadessy is described as a simple, yet robust constrained non-linear hydraulic gradient network reticulation design method (Ilemobade and Stephenson 2003:199), which optimizes pipe sizes within prescribed constraints (Ilemobade and Stephenson 2006). Wadessy is made up of a suite of computer programs which facilitate the optimal design, planning and daily operation of small WDSs (Ilemobade and Stephenson 2003). It is a software that was developed to assist with the facilitation of technical planning and operations of small WDSs. Ilemobade and Stephenson (2006) described Wadessy as an efficient non-linear algorithm which can determine the WDN's hydraulic gradient thereby optimizing the pipe sizes within the given limits.

The optimization algorithm used in Wadessy is based on the concept that a hypothetical hydraulic gradient for a WDN that is hydraulically balanced exists, and when achieved iteratively it produces optimal pipe sizes with an optimal flow relation between each pipe. To establish an optimal WRN design, an iterative hydraulic

simulation-optimization algorithm is employed. The efficient hydraulic simulation depends on modelling the WRN using the equations of continuity, conservation of energy and hydraulic head loss relation and determining the unknown variables (Q_i) or residual pressure heads at nodes. This can be achieved by using the Newton-Raphson iterative procedure on equations generated using the nodal formulation method (Ilemobade and Stephenson 2006).

The basic equations of continuity, conservation of energy and pressure difference are used in modelling the network and an optimum solution is obtained after employing an iterative simulation-optimization algorithm. To solve the optimization problem, optimization equations for cost and reliability need to be formulated and solved. The objective of Wadessy's suite of programs is to minimize capital and recurrent costs of the major components of WDS subject to certain constraints (Ilemobade and Stephenson 2003). Based on this, the system can be modelled as follows:

$$Minimize\ C_{\ water\ distribution\ system} = (C_{WRN} + C_{\ pump\ and\ tank\ sub\text{-}system}) \quad (2.9)$$

Where C is the cost, and this can be further expanded as below,

$$C_{\ pump\ and\ tank\ sub\text{-}system} = (C_{\ pump\ installation} + C_{\ pump\ operation} + C_{\ pumping\ mains} + C_{\ tank\ storage}) \quad (2.10)$$

The minimization of the WRN is a function of hydraulic gradient and pipe diameter whilst that of the pump and reservoir sub-system is a function of pumping mains flow capacity and pipe diameter. Finally, the WDS objective function can be solved subject to constraints of pipe diameter, pumping mains flow capacity, node pressure head and minimum system cost.

Wadessy's User Manual states that Wadessy comprises modules that can perform design optimization and costing of new WRN, upgrade optimization and costing of existing WRN, steady-state and extended-state simulation, optimization and costing of pump and reservoir systems and single-value and time-series demand input and modelling. The output from the simulation with Wadessy include pipe flows, pumping heads, head losses for both pipes and valves, flow orientation, nodal pressure, and associated pipe costs. After the simulation, the pipes may be optimized to establish optimal pipe sizes and the cost thereof (Ilemobade and Stephenson 2006).

Wadessy's, advantages include its ease of use for the Southern Africa situation, the substantial savings in design time, and capability to produce comparable results with other tools (Ilemobade and Stephenson 2006). Currently, there are other alternatives such as evolutionary algorithms, which have been described as extremely effective in providing various feasible solutions for both small and large systems. Their disadvantage when compared to Wadessy is that they require a large number of runs to ascertain an optimal solution because they utilize several parameters and also require considerable design skills to set out and optimize a WDN due to the multiple variables involved (Ilemobade and Stephenson 2006).

While many of the optimization methods discussed in this literature review, require considerable skills to use, Wadessy is presented as a simple tool which presents opportunities for especially Southern Africa's designers who generally lack the required skills because of training challenges. It is against this background that Wadessy was chosen for this investigation. There are other advantages presented by Ilemobade and Stephenson (2006) including the fact that Wadessy is customized for Southern African conditions, capital and pump operating costs are determined in a separate program and then inserted into the objective function after calculation and that Wadessy determines the dummy hydraulic gradient, S_0 by rearranging the objective function.

3. CASE STUDIES OF SIMULATION – OPTIMIZATION OF WRNs

In this chapter, case studies of standard WDS benchmark problems used to test various optimization algorithms are presented. Benchmark networks are used to compare and test network optimization methods and have features and problems typical of those found in real networks (Farmani *et al*. 2005), and they provide a common testbed for new optimization algorithms (Sangroula *et al.* 2022). According to De Corte and Sörensen (2013), the most frequently used benchmark networks for case studies of optimization of WRNs are the two-loop network, the New York City tunnels network, and the Hanoi network. Mala-Jetmarova *et al.* (2018) highlighted that most of the optimization studies solve a single objective least-cost design problem using a small number of available benchmark networks such as Hanoi network, New York City tunnels and two-loop network, to test the optimization method. The other popular benchmark networks used in research is the Anytown WDS which was presented by Walski *et al*. (1987) for the hypothetical town of Anytown and the two-reservoir Gessler network by Gessler (1985). Also included in this section is the Selebi-Phikwe (SP) WRN, on which the Wadessy design tool was employed twice by Ilemobade and Stephenson (2003; 2006). A summary of some of the frequently used benchmark networks is shown in Table 3.1 including their usage based on Mala-Jetmarova *et al.* (2018)'s review of 107 publications on optimization of WDSs.

Table 3.1 : Benchmark networks for case studies of optimization of WRNs

Network	**Originator/ First optimization**	**Network usage count**
The two-loop network	Alperovits and Shamir (1977)	40
Hanoi network	Fujiwara and Khang (1990)	55
The New York City tunnels	Schaake and Lai (1969)	42
The Anytown water distribution system	Walski *et al*. (1987)	15
Two-reservoir Gessler network	Gessler (1985)	1
Selebi-Phikwe network	Ilemobade and Stephenson (2003)	1

3.1 Two-loop network

This network is a two-loop network supplied from a constant head reservoir by gravity (Alperovits and Shamir 1977). The two-loop network was presented by Alperovits and Shamir to validate the (non-linear programming) LPG model (Ilemobade and Stephenson 2003). The network consists of eight pipes, six junctions and a reservoir. The layout of the pipe network arranged in two loops is shown in Figure 3.1. The minimum node head requirement for all nodes is 30m. The length of each pipe is 1000m and the Hazen-Williams coefficient is assumed to be 130 for all pipes in the network (Abebe and Solomatine 1998; Geem 2006).

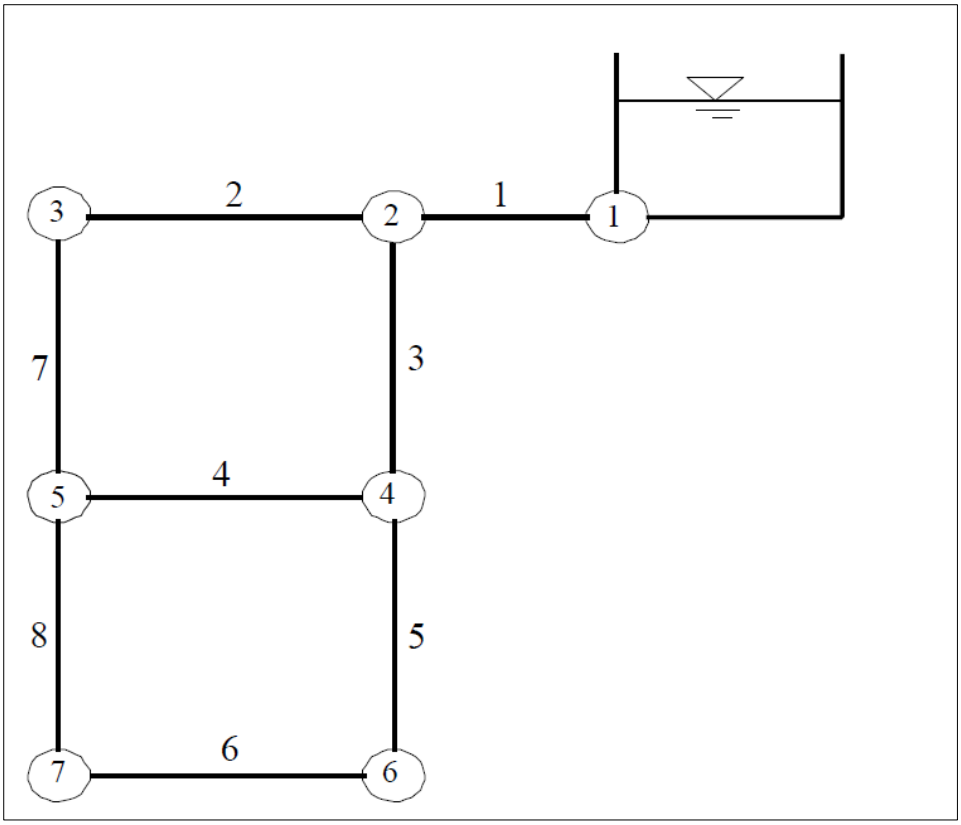


Figure 3.1 : Two-loop network (Abebe and Solomatine 1998)

Abebe and Solomatine (1998) used a global optimization tool with various random search algorithms and a network simulation model to solve the problem. The random search algorithms for this problem included Controlled Random Search (CRS2), CRS4, Genetic Algorithm and Adaptive Cluster Covering with Local Search (ACCOL) (Abebe and Solomatine 1998). The objective function to be minimized by the optimization algorithms was the network cost. A penalty cost was added to the actual cost of the network based on the minimum head constraint. The exercise by Abebe and Solomatine (1998) yielded better solutions enabling a choice between accuracy

and required computer time. All algorithms stopped near the optimum, with feasible and single diameter solutions in all cases.

Geem (2006) also applied the harmony search algorithm to the two-loop network problem. The meta-heuristic harmony search algorithm copies a jazz musician's improvisation behaviour, which can be interpreted as an optimization process. The procedure for the harmony search method consists of memory consideration, pitch adjustment and random selection (Geem 2006). The results obtained using the harmony search based model by Geem (2006) were compared with the results of other methods used on the two-loop network by Alperovits and Shamir (1977), Goulter *et al* (1986) and Kessler and Sharmir (1989), Savić and Walters (1997), and Cunha and Sousa (1999). Optimum solutions obtained by Savić and Walters (1997), and Cunha and Sousa (1999) were similar to the solution obtained by Geem (2006). However, the solution by Geem (2006) had better results than the solution by Alperovits and Shamir (1977), and Goulter *et al* (1986) but was more expensive than the solution by Kessler and Sharmir (1989).

Ilemobade and Stephenson (2003) studied the operation of the two-loop network under peak and night flow conditions. They used the Wadessy optimization tool with the objective emphasizing the reduction to minimum, the capital and recurrent costs of the major components of the WDS. After running the optimization procedure from several starting network designs, the optimal solution was realised. The optimization process allowed a 1.5% deviation from the minimum residual pressure of 30m. The results obtained by Wadessy realised a cost saving of 1.91% from Alperovits and Shamir's (1977) solution and 2.24% from Farmani *et al.* (1999) solution (Ilemobade and Stephenson 2003). The comparison of optimisation results for the two loop network based on various researchers using different optimization methods is shown inTable 3.2.

Table 3.2 : Cost comparison of published solutions for two-loop network

Item	Description	Total cost
1	Alperovits and Shamir (1977)	$479 525
2	Goulter *et al*. (1986) (as cited in Abebe and Solomatine (1998))	$435 015
3	Savic and Walters (1997)	$420 000
4	Abebe and Solomatine (1998)	$424 000
5	Geem (2006)	$419 000
6	Shende and Chau (2019)	$419 000
7	Sangroula *et al*. (2022)	$419 000

3.2 Hanoi network

The Hanoi network (shown in Figure 3.2) is gravity fed from a reservoir with a 100m fixed head (Fujiwara and Khang 1990, as cited by Perelman *et al.* 2013, Boindala and Ostfeld 2022). It consists of 32 demand nodes, 34 pipes organized in 3 loops, with the pipe lengths varying from 100m to 3500m, with a Hazen-Williams coefficient of 130 (Geem 2006). The minimum head required at all nodes for this network is 30m. There are six possible pipe diameters that the network needs to be designed with.

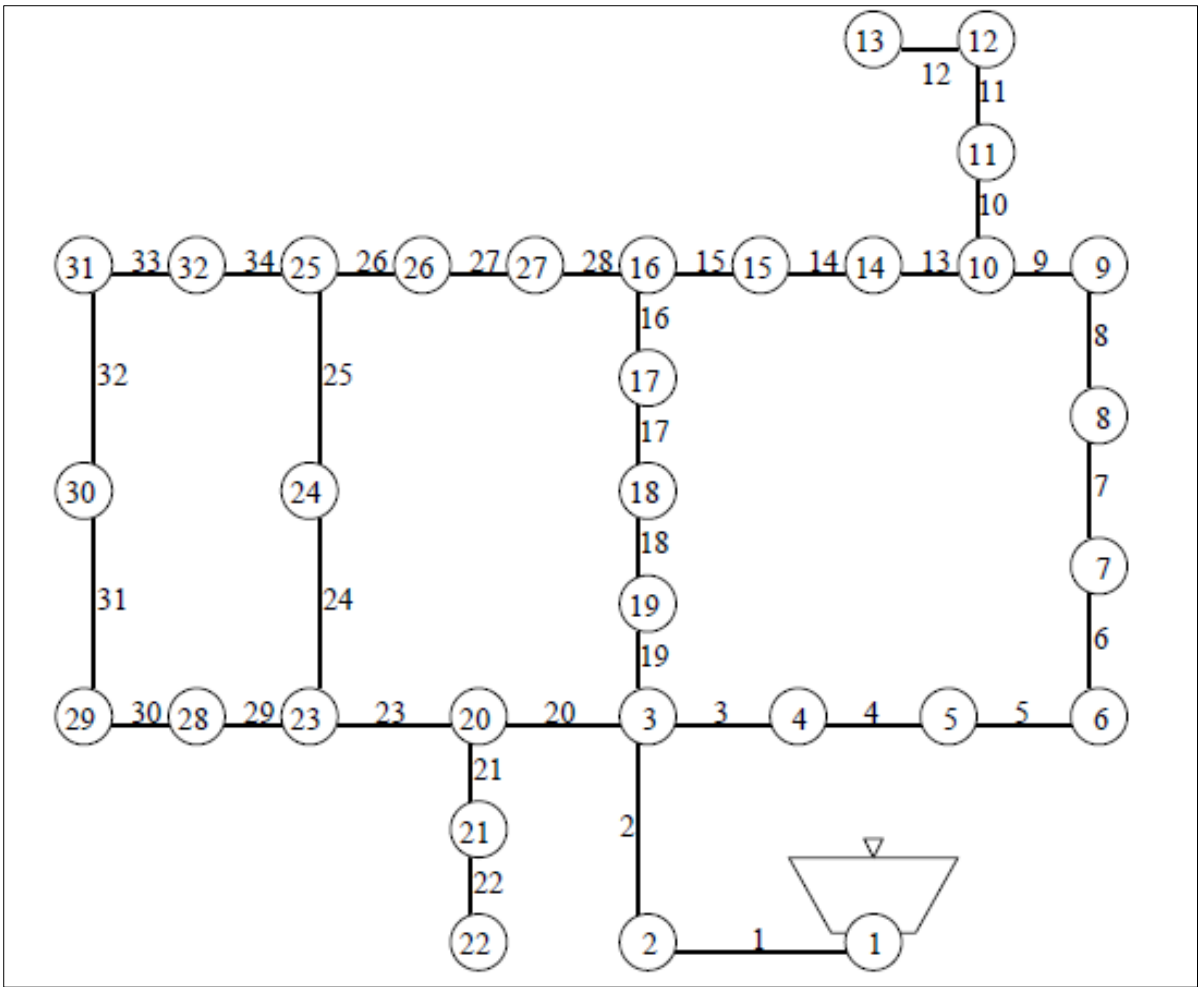


Figure 3.2 : Graphical representation of Hanoi WDS (Geem 2006).

The network was initially optimized by Fujiwara and Khang (1990), utilizing the two phase non-linear programming method. Thereafter, several case studies have used the Hanoi network for various optimization techniques.

Boindala and Ostfeld (2022) used the self-adaptive multi-objective cuckoo search algorithm (SAMOCSA) combined with an inbuilt nonlinear optimization algorithm in MATLAB named “fmincon” to solve the robust multi-objective WDS design optimization problem. SAMOCSA is described as an improved version of the multi-objective cuckoo search that adapts the algorithm’s exploration and exploitation which governs the parameters at every iteration (Boindala and Ostfeld 2022). The main objective of the study by Boindala and Ostfeld (2022) was to investigate the effect of demand uncertainty and the correlation on the WDS designs in a multi-objective framework. The results inferred from the Hanoi case study by using SAMOCSA indicated that positively correlated demand patterns require higher cost designs to maintain even low resilience in the network. Furthermore, the results

showed that when comparing higher topological resilience networks and inherently low topological resilience networks, the effect of uncertainty in demand is higher for the latter.

Abebe and Solomatine (1998) also carried out optimization on the Hanoi network using the same algorithms they used for the two-loop network in section 3.1 i.e., global optimization tool with various random search algorithms. The CRS2 and CRS4 algorithms failed because they were designed for continuous variables. However, the genetic algorithm and ACCOL moved the search towards the global minimum. ACCOL reported a solution that is slightly more expensive (11%) than that of Genetic Algorithms even though it required 3 to 5 times more function evaluations.

The harmony search algorithm was also applied to the Hanoi WDS by Geem (2006), and a comparison was done with results from Fujiwara and Kang (1990), and Savić and Walters (1997). Fujiwara and Kang (1990) used the nonlinear programming gradient (NLPG) and local improvement method whilst Savić and Walters (1997, as cited in Geem 2006) used the genetic algorithm. Overall Geem (2006)'s method obtained a cheaper solution than the other methods as shown in Table 3.3:.

Table 3.3: Cost comparison of published solutions of Hanoi network

Item	**Description**	**Total cost**
1	Fujiwara and Kang (1990)	$6 320 000
2	Savić and Walters (1997)	$6 073 000
3	Abebe and Solomatine (1998)	$7 006 000
4	Geem (2006)	$6 056 000
5	Shende and Chau (2019)	$6 081 087
6	Boindala and Ostfeld (2022)	$6 584 000
7	Sangroula et al. (2022)	$6 081 115

3.3 New York City tunnels water supply system

Several researchers used the New York City tunnels network as a benchmark for pipe network optimization techniques (Ilemobade and Stephenson 2006). Schaake and Lai first presented the New York City network, shown in Figure 3.3 (Geem 2006). The network consists of 20 nodes, 21 pipes and 1 loop, and is fed by gravity from a reservoir with a 91.44m fixed head. The initial objective of the problem was to reinforce the water supply by adding new pipes parallel to existing ones because the existing network could not satisfy the pressure head requirements at certain key nodes (nodes 16 to 20) (Geem 2006). The New York City tunnels is the most frequently applied test networks considering system strengthening (Mala-Jetmarova *et al.* 2018).

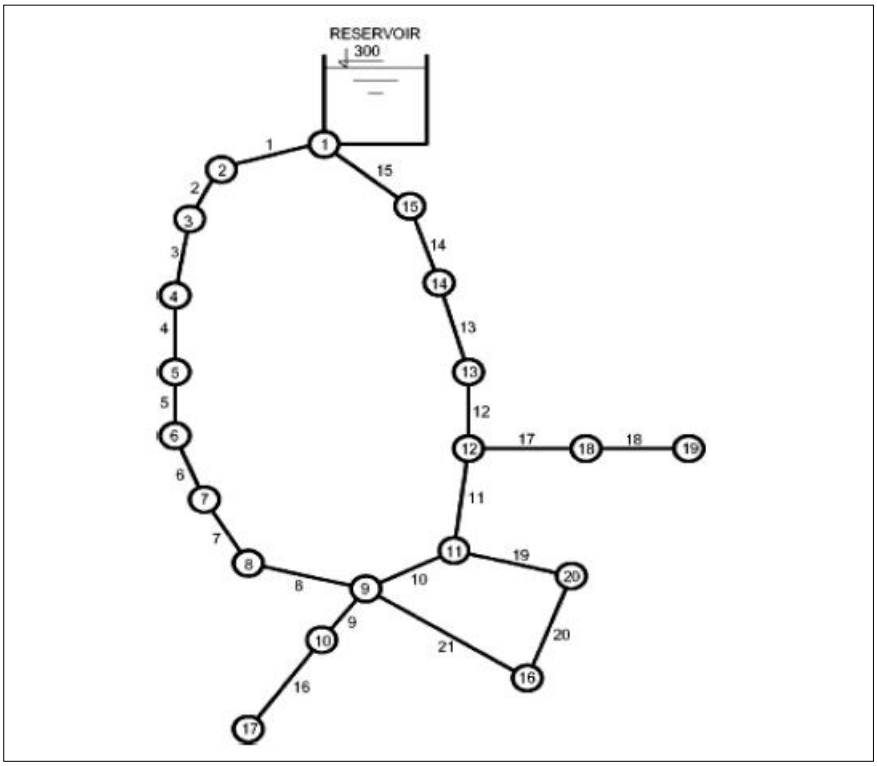


Figure 3.3 : New York City tunnels network (Ilemobade and Stephenson 2006)

Geem (2006) optimized the New York City tunnels network using, harmony search method and the results were compared with the results previously obtained by Schaake and Lai (1969), Savić and Walters (1997), and Cunha and Sousa (1999) and were found to be the most favourable (Geem 2006). Schaake and Lai (1969) used mathematical programming whilst Savić and Walters (1997) employed genetic algorithm-based model, and Cunha and Sousa (1999) used a simulated annealing-based model.

Wadessy's design tool was calibrated using the New York City water supply tunnels problem. Ilemobade and Stephenson (2006) stated that the objective of previous studies on the New York City Tunnels network was to determine the most economic design for the reinforcement of the WDN by additions to the existing system. This is congruent with the overall objective of Wadessy's suite of programs which seek to minimize costs (both capital and recurrent) of the major components of a WDS. Ilemobade and Stephenson (2006) started the optimization process from several starting designs so that several optimized solutions could be generated and ensure the good quality of the optimal solution. The Wadessy design tool produced comparable pipe costs to those previously obtained for the New York City water supply tunnels problem as shown in Table 3.4.

Table 3.4 : Cost comparison of published solutions of New York City tunnels network

Item	**Description**	**Total cost**
1	Schaake and Lai (1969)	$78 090 000
2	Savić and Walters (1997)	$37 130 000
3	Cunha and Sousa (1999)	$37 130 000
4	Ilemobade and Stephenson (2006)	$39 620 000
5	Geem (2006)	$36 660 000

3.4 Anytown water distribution system

The Anytown WDS was presented by Walski *et al*. (1987) for the hypothetical town of Anytown. The objective of the Anytown network problem was to establish the most economically effective design to reinforce the existing WDS to meet future demands. The proposed design was supposed to consider pumping costs as well as capital expenditure. There were several options for the new design which include new pipes, cleaning and lining the existing pipes, duplication of existing pipes, construction of new pumping stations, upgrading existing pumping station, and provision of new tanks (Prasad and Tanyimboh 2008:457). The Anytown WDS is depicted by the pipe network in Figure 3.4. The network comprises 19 junctions, three tanks and a pumping station with three identical pumps. Even though this network does not contain all features of a WDN, it acts as a challenging benchmark

for optimization models that consider real system features like pump, tank sizing and location (Prasad and Tanyimboh 2008).

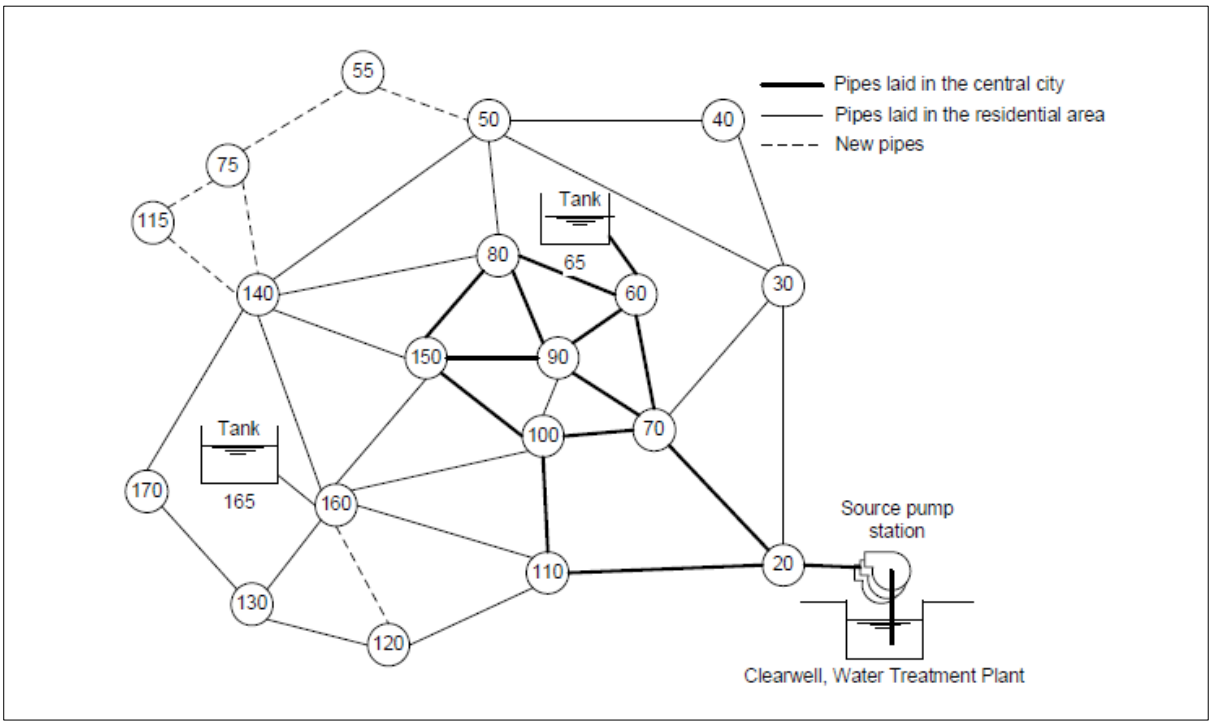


Figure 3.4 : The Anytown WDS (Prasad and Tanyimboh 2008)

The Anytown problem was originally solved by the participants at the “Battle of the Network Models" workshop (Walters *et al*. 1999) and has since been examined by several other researchers using different optimization methods (Prasad and Tanyimboh 2008).

Walters *et al*. (1999) applied the Structured Messy Genetic Algorithm (SMGA) model to the benchmark “Anytown” problem. The SMGA model for optimising WDN rehabilitation was expanded to include pipe rehabilitation decisions, pumping installations and storage tanks as variables. In their study (Walters *et al.* 1999) reformulated the problem in terms of minimising cost and maximising benefit, by treating the benefit as the reduction in predicted hydraulic deficiencies. Their solution was 4-5% cheaper than any previously published solutions to this problem (Walters *et al.* 1999).

The Anytown network was also used as an example by Farmani *et al.* (2005) to investigate the application of multi-objective evolutionary algorithms in identifying the pay-off characteristics between total cost and reliability of a WDS. For this study, the problem was posed as a multi-objective optimization problem with objectives to minimize the costs and maximize the resilience of the network. The design variables

were the same as those used by Prasad and Tanyimboh (2008). The results showed that the pipe network design varied among the solutions on the Pareto front depending on the level of reliability achieved and investment on pipe capital costs.

Prasad and Tanyimboh (2008) performed an entropy-based design of the Anytown WDS. The main objective of their study was to show that, many feasible solutions could be obtained along the Pareto front using the tank design methodology and the surrogate reliability measure could be used effectively to improve reliability of multi-source networks. They formulated a multi-objective model, namely NSGA-II with minimization of infrastructure cost and maximization of surrogate reliability measures that relied on the entropy concept considered as objectives. The constraints of the model included pressure constraints and other operational constraints such as recovery of water levels in tanks, the use of full capacity of existing and new tanks (Prasad and Tanyimboh 2008). The findings of this study indicated a better performance of tank sizing approach and improved reliability of networks obtained along the Pareto front. A comparison of pipe costs for the solutions obtained by various researchers for the Anytown network are shown in Table 3.5.

Table 3.5 : Cost comparison of published solutions of Anytown network

Item	**Description**	**Total cost**
1	Walski *et al*. (1987)	$12 300 000
2	Murphy et al. (1994)	$11 400 000
3	Walters *et al*. (1999)	$10 910 000
4	Farmani *et al.* (2005)	$13 350 000
5	Prasad and Tanyimboh (2008)	$11 650 000
6	Siew *et al.* (2016)	$10 310 000

3.5 Two-reservoir Gessler network

Gessler (1985) proposed the two-reservoir network to demonstrate the solution of optimization problems by partial enumeration, such as the Wadiso approach. The design problem of the Gessler pipe network involved an expansion to an existing

looped, gravity fed WDS. The network consists of one reservoir, one tank, fourteen pipes comprising five new pipes and nine existing pipes. The layout of the pipe network (both existing and proposed) is shown in Figure 3.5.

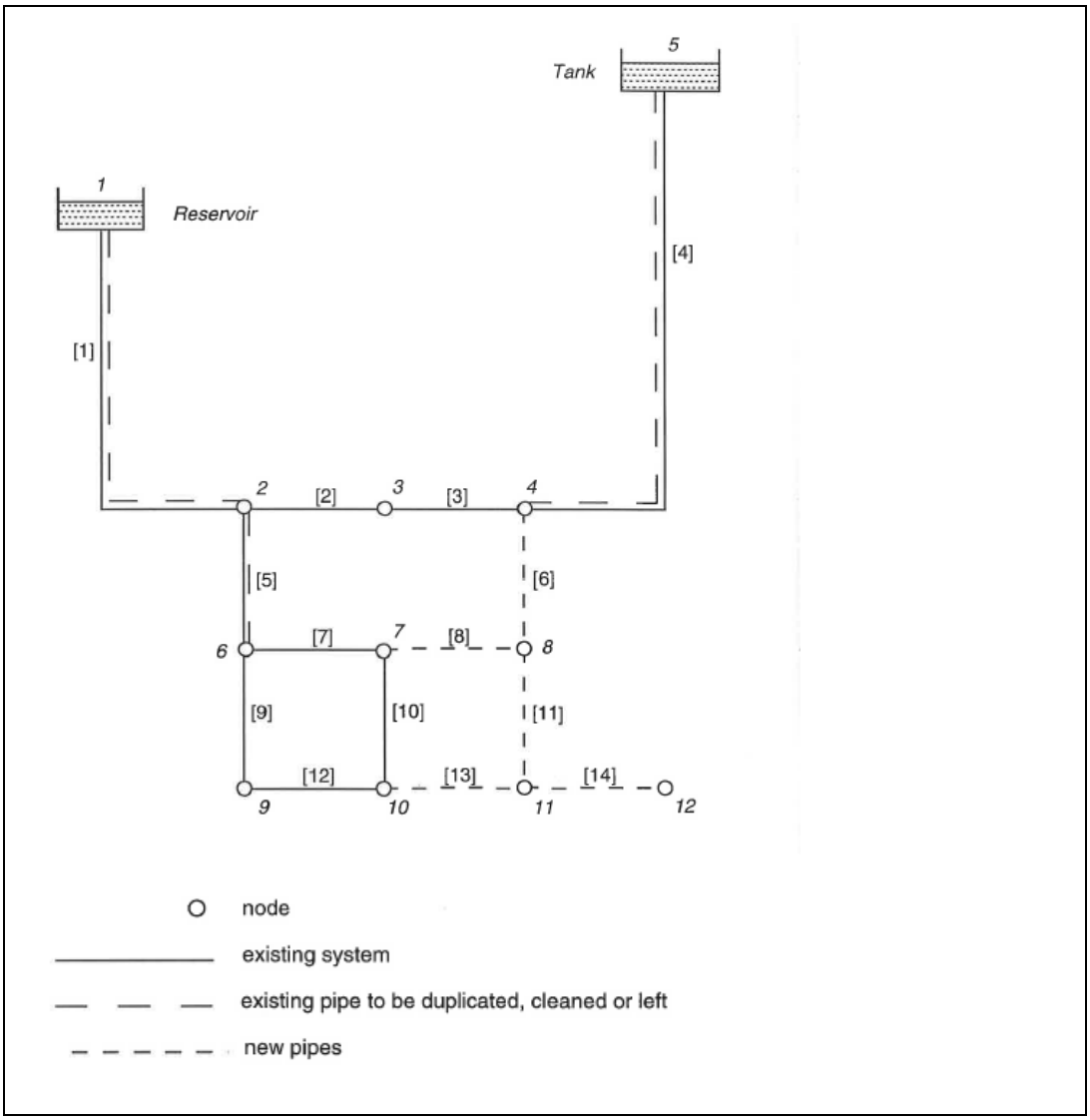


Figure 3.5 : The Gessler network layout

The expansion of the network required sizing of five new pipes and upgrading (cleaning, duplication or 'do nothing') of three existing pipes. The solution space was initially reduced to 900 combinations by grouping the pipes into five groups in which pipes had similar candidate design parameters shown in Table 3.6.

Table 3.6 : Pipe groups (Gessler 1985)

Pipe group	Pipes in group	Candidate design parameters (inches)
1	[1] and [5]	leave, clean, dup 12, dup 14, dup 16
2	[4]	leave, clean, dup 12, dup 14, dup 16
3	[6] and [11]	8, 10, 12
4	[8] and [13]	6, 8, 10
5	[14]	6, 8, 10, 12

A further reduction of combinations was achieved by eliminating the 6-inch pipe for pipe group 5 and not considering the 'leave and clean' alternatives for pipe group 3. Finally, 405 combinations were enumerated with about 94% of the combinations excluded by using the 'cost filter' and 'size filter". The solution obtained by partial enumeration ranked number 23 on the list of the best solutions to the two-reservoir problem (Murphy 1997) which was 4.76% higher in cost than the global optimum.

3.6 Selebi-Phikwe network

3.6.1 Existing Selebi-Phikwe WRN performance check

The Wadessy software was used to analyse the existing SP WRN (depicted in Figure 3.6) to determine the performance of the existing network on the addition of three new residential developments (Ilemobade and Stephenson 2003).

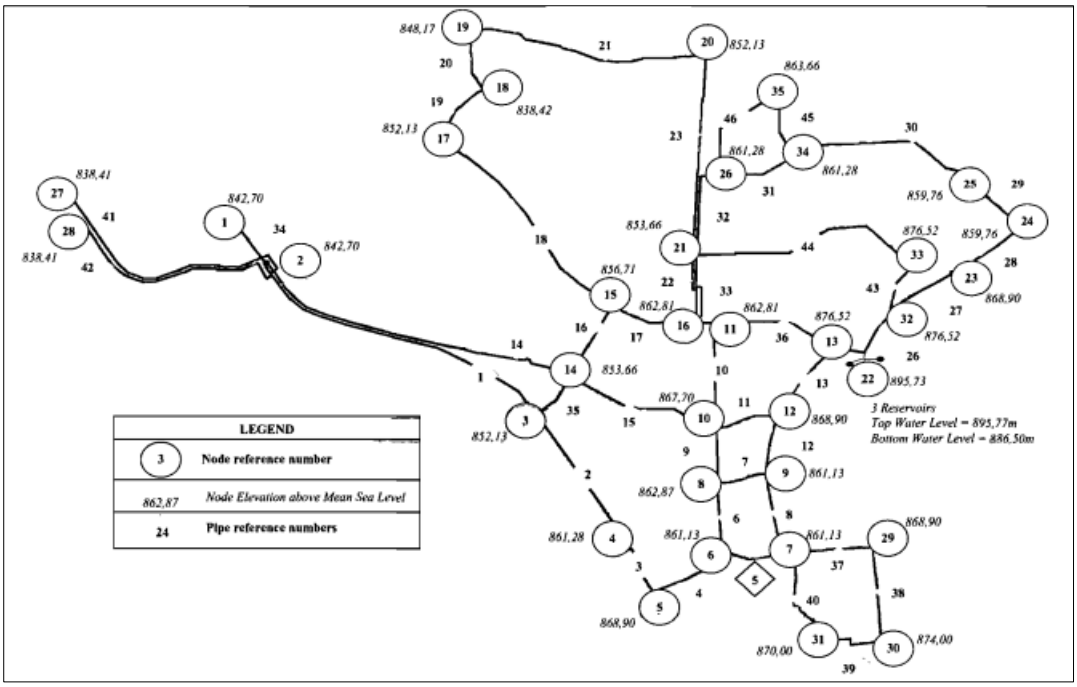


Figure 3.6 : Skeletonized existing SP WRN (Ilemobade and Stephenson 2003)

3.6.2 Selebi-Phikwe WRN upgrade

Wadessy's design tool was also employed on the SP WRN upgrade project. The SP WRN upgrade was meant to address the low pressures at certain nodes within the existing WRN and provision of potable water through the existing SP WRN to Mekoro, Block A and Block B as shown in Figure 3.7. A consulting firm had initially attempted to optimize the SP WRN upgrade by trial and error before the use of Wadessy's design tool (Ilemobade and Stephenson 2006). With the utilization of Wadessy in the SP WRN upgrade, a cost saving of 62% was realised.

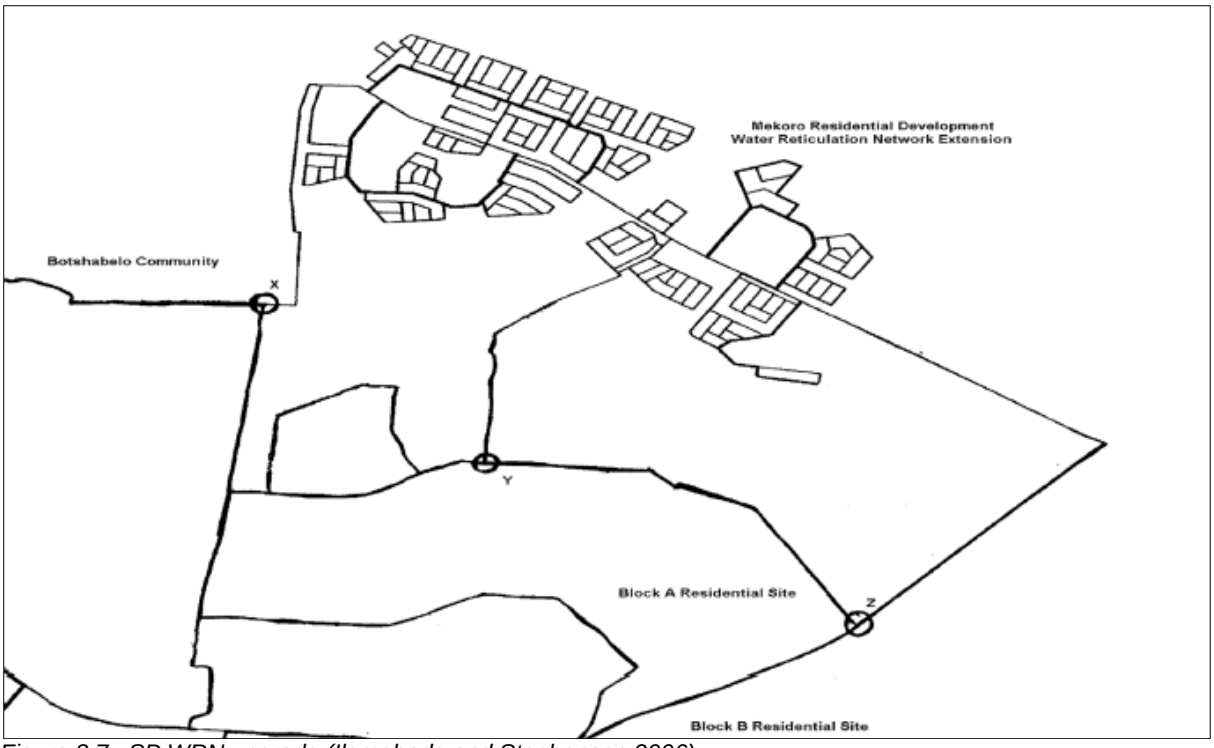


Figure 3.7 : SP WRN upgrade (Ilemobade and Stephenson 2006)

4. STUDY AREA, METHODOLOGY AND DATA COLLECTION

4.1 Description of the Study Area

The study area is the Tswelopele Village, which is found in Hekpoort Rural Township under Mogale City Local Municipality in the South African province of Gauteng. The area encompasses gentle sloping terrains to flat lands along the foot of the Magaliesberg Ridges within valley lines and surrounded by farms (SFCE 2015).

4.1.1 Location

The Tswelopele Village coordinates are 25°55'49.16"S latitude and 27°30'14.93"E longitude, near the R677 road which branches off the R24 road to Rustenburg. It is situated on the southern slopes of the Magaliesberg Mountain. The locality of the study area is shown in Figure 4.1.

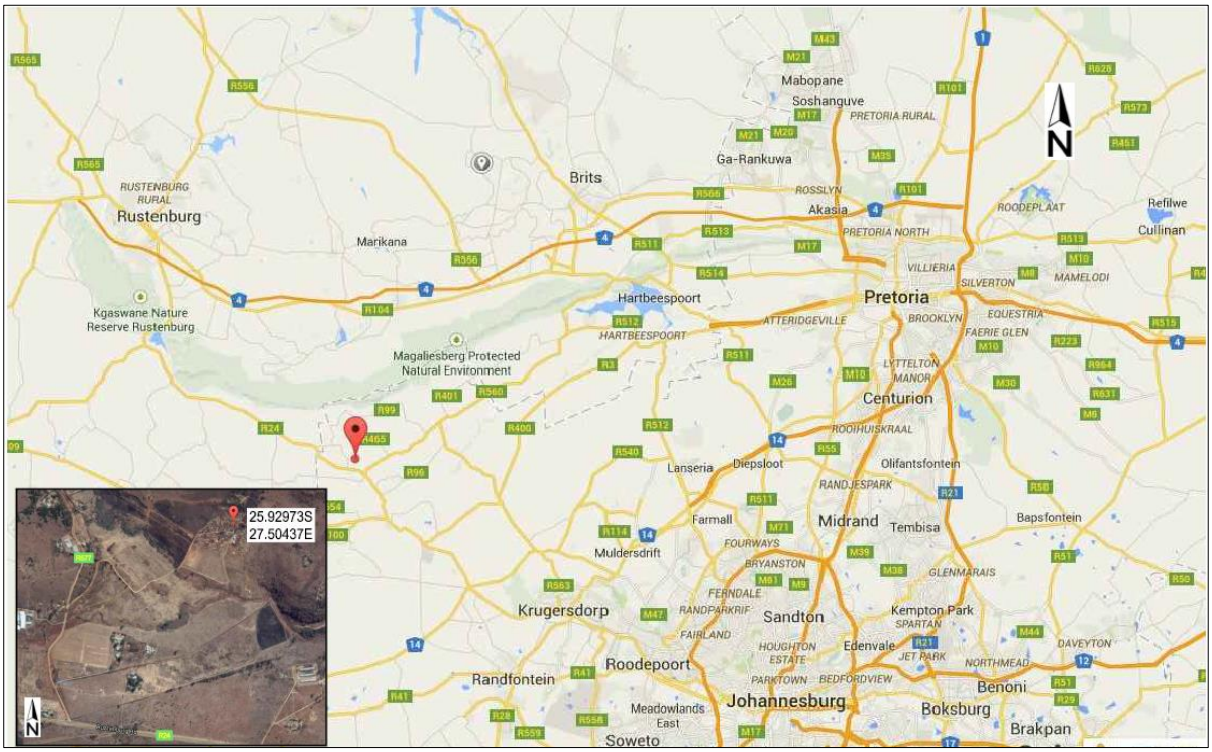


Figure 4.1 : Locality Map (Google Maps 2015)

4.1.2 Population Characteristics

No population data of the study area could be obtained. Therefore, a physical count of households in the Village was done, and the count indicated that there were 89 households in Tswelopele Village. For the purposes of this research, an estimate of 6 people per household was used (CSIR, 2000), bringing the total population to 534 people. No major economic activities take place in the Village besides a community

garden which is supplied water by a different water network. Most of the residents in the Village work in the nearby farms.

4.1.3 Climate and Rainfall Distribution

Mogale City Local Municipality is located in the Southern Hemisphere. The climate of the area under Mogale City Local Municipality is warm and temperate. The summers receive good amounts of rainfall, but the winters only receive small amounts. The average annual temperature of the area is 16.9°C and about 751mm of precipitation falls annually (Climate Data 2023).

4.1.4 Tswelopele Village Water Supply

The Mogale City Local Municipality, with its seat in the nearby town of Krugersdorp, is responsible for basic water supply provision and other service delivery issues in Tswelopele Village. However, since Tswelopele Village is an informal settlement, it is isolated from the water supply grid, which mainly serves the urban areas of the Municipality. To connect informal settlements to existing water supply networks, may have a substantial impact on the WDS (DHS 2019), since they are normally located far from the system.

The Village's main supply of potable water is the borehole located nearby. The recommended yield of the borehole is 1.5l/s for 8hr/day. The borehole which was previously abandoned was resuscitated, equipped, and powered in 2015 during the rehabilitation project. Borehole yield tests that were conducted for 24hrs indicated a yield rate of 1.22L/sec, and final drawdown of 12.68m was recorded (SFCE 2015).

The water is pumped by a solar powered pump to the storage on top of the hill next to the Village, through a 50mm HDPE pipeline. The distance from the pumphouse to the storage is approximately 500m. The three tanks comprising the Village's storage have a total capacity of 30,000litres. Before the rehabilitation in 2015, there was only one 10kL steel tank. Two 10kL HDPE tanks were installed in 2015 as part of the rehabilitation project to augment the water storage. The main purpose of the tanks is to balance out peaks in demand and to also provide emergency storage (DHS 2019). The use of intermediate tanks also provides the added benefits of reducing pressure.

The existing distribution system is a gravity flow reticulation system that comprises of 50mm HDPE, Class 9 pipes for both the mainline and branches. The system was rehabilitated in 2015 and financed under the CRDP programme. However, there

have been ongoing water supply challenges in the Village which are suspected to be due to design issues or leakages. There are some houses that are receiving water at below minimum pressures, some not receiving water at all and some are not connected to the existing WDN because when the rehabilitation was implemented in 2015, it only targeted the households that were already connected to the network. Only 54 households were reconnected to the rehabilitated WDN, and 35 households were not connected. All the water connections in the Village are yard connections, and the Village utilizes dry sanitation (SFCE 2015). The DHS (2019) indicates that yard connections usually satisfy the requirements of basic level of service in terms of free basic service policy. The existing water distribution network is shown by green lines in Figure 4.2.

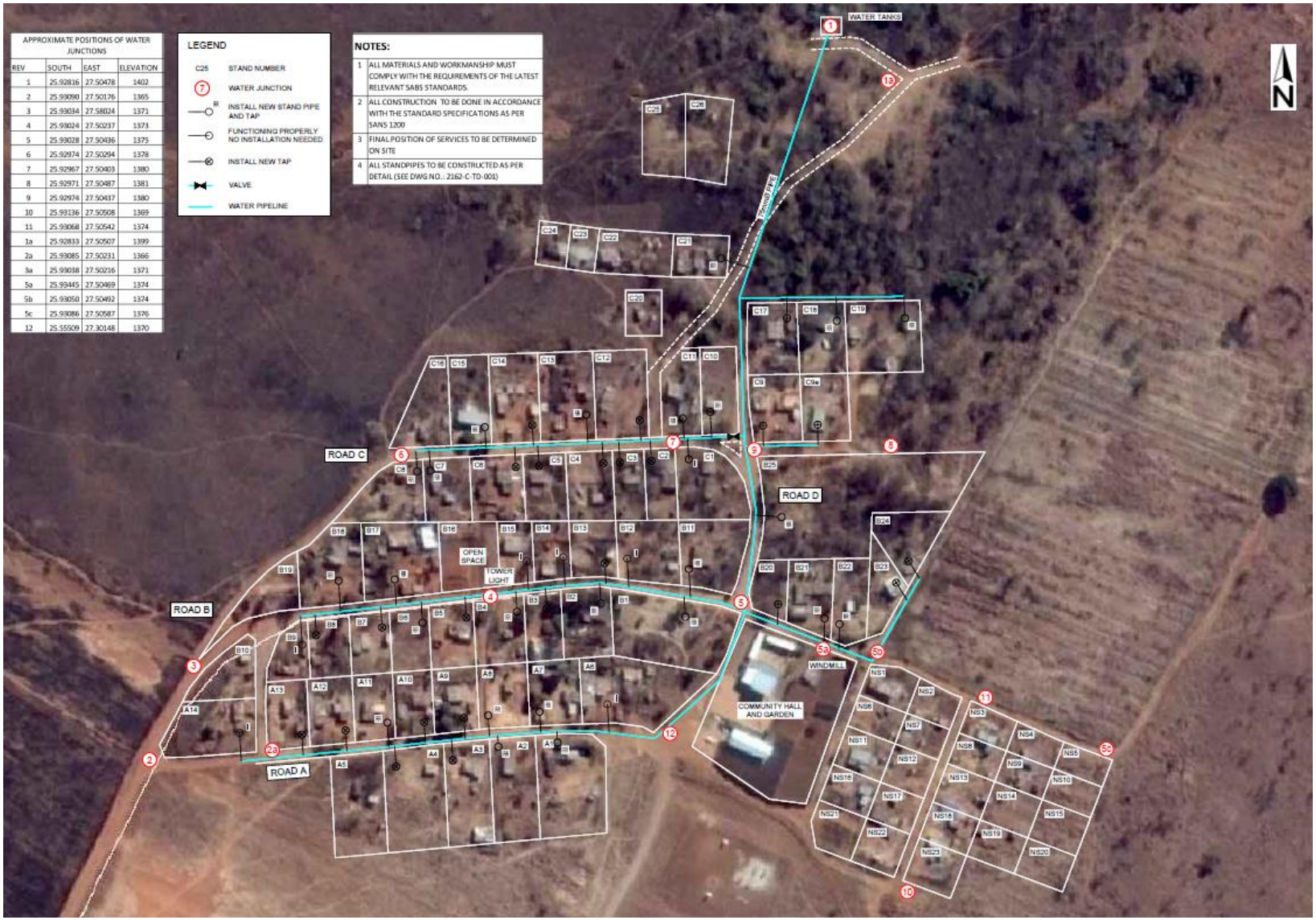


Figure 4.2 : Existing water distribution network (SFCE 2015)

4.2 Data Collection

Data was received from both primary and secondary data sources. Primary data was gathered from various sources including the local community, Mogale City Local Municipality and Department of Rural Development and Land Reform. This was accomplished through site visits and discussions with the stakeholders. Pictures of various WDN components in Tswelopele Village such as tanks, pumphouse and standpipes are shown in Figure 4.3 to Figure 4.8 (SFCE 2015).



Figure 4.3 :Solar powered borehole



Figure 4.4 : Pumphouse



Figure 4.5 : Solar panels powering the borehole



Figure 4.6 : Water main line during construction



Figure 4.7 : Tanks



Figure 4.8 : Yard tap

The secondary data was collected from design, tender and contract documents (SFCE 2015) of the rehabilitation project undertaken in 2015. The information includes households' information, topographical data, climate data and existing distribution network parameters such as elevations, pipe diameter, pipe length and pipe material.

4.2.1 Water Quality

The DHS (2019) indicates that potable water must comply with South African National Standard-Drinking Water, Part 1: Microbiological, physical, aesthetic, and chemical determinants (SANS 241-1) which guides the requirements of health-based water quality without considering the source and treatment process used. Potable water must be free of contaminants and pollutants that could affect human health, and must be clear, tasteless and odourless (DHS 2019). For the Tswelopele Village, a water sample was taken at the end of the borehole constant rate test and sent to laboratory for quality testing. The water quality test yielded Class 0 results as per SANS 241-1 classification as shown in Table 4.1.

According to the Water Research Commission (WRC), class 0 results indicate ideal water suitable to be used as drinking water by all users without the need for treatment.

Table 4.1 : Water quality results (SFCE 2015)

ANALYSES	UNIT	Tswelopele 15-02-2014	CLASSIFICATION Class 0	Class 1	Class 2	Class 3	Class 4
Ph		6,90	5.5 - 9.5	4.5 - 10	4 - 10.5	3 - 11	< 3 or > 11
Conductivity	mS/m	40,60	< 70	70 - 150	150 - 370	370 - 520	> 520
TDS	mg/l	263,90	< 450	450 - 1000	1000 - 2400	2400 - 3400	> 3400
Nitrate (N)	mg/l	1,40	< 6	6 - 10	10 - 20	20 - 40	> 40
Fluoride	mg/l	0,14	< 0.7	0.7 - 1	1 - 1.5	1.5 - 3.5	> 3.5
Sulphate	mg/l	11,16	< 200	200 - 400	400 - 600	600 - 1000	> 1000
Chloride	mg/l	7,70	< 100	100 - 200	200 - 600	600 - 1200	> 1200
Carbonate		0,00					
Bicarbonate		205,00					
Total Hardness	CaCo3	187,56	< 200	200 - 300	300 - 600	> 600	
Ca - Hardness		64,93					
Mg - Hardness		122,63					
Calcium	mg/l	25,97	< 80	80 - 150	150 - 300	> 300	
Magnesium	mg/l	29,91	< 70	70 - 100	100 - 200	200 - 400	> 400
Sodium	mg/l	12,87	< 100	100 - 200	200 - 400	400 - 1000	> 1000
Potassium	mg/l	1,82	< 25	25 - 50	50 - 100	100 - 500	> 500
Iron	mg/l	0,02	< .5	.5 - 1	1 - 5	5 - 10	> 10
Manganese	mg/l	0,16	< .1	.1 - .4	.4 - 4	4 - 10	> 10
WATER CLASS		Class 0					

4.2.2 Population Projections

The focus when developing a WDN is to meet future demands in an efficient manner. Therefore, the design of a WDN is based on projected population of the subject area over a certain period (Anisha *et al*. 2016). The system must be designed based on the population at the end of the design period. Parameters used for estimation of population and water demand are given in Table 4.2.

Table 4.2 : Parameters for estimation of population and water demand

Item	Parameter	Value
1	Population base year	2015 (available data)
2	Design period	20years
3	Growth rate	2.5%
4	Number of households	89
5	Number of people per household	6
6	Total population	534
7	Demand per capita	80L/c/d
8	Peak Factor	1.5 (based on low-income housing, DHS (2019)).
9	Reticulation losses factor	25% (anticipated losses, DHS, (2019))

The future population of Tswelopele Village was projected (Table 4.3) using the geometric increase method. It assumes that the percentage increase remains constant and applies the following formula :

$$P_n = P_o[1 + (r/100)]^n \quad \text{.....(4.1)}$$

Where,

P_n = population after 'n' years,

P_o = initial population,

n = number of years between P_o and P_n.

r = Increase in population in a year/Population at the beginning of the year.

Table 4.3 : Population Growth Estimate

Year	Population	Growth Factor (2.5 %)
2015	534	0.025
	548	0.025
	562	0.025
	577	0.025
	592	0.025
2020	607	0.025
	623	0.025
	639	0.025
	655	0.025
	672	0.025
2025	689	0.025
	707	0.025
	725	0.025
	744	0.025
	763	0.025
2030	783	0.025
	803	0.025
	824	0.025
	845	0.025
	867	0.025
2035	889	0.025

The table shows the population for the base year 2015, and the other years until ultimate year, 2035. The full calculation of the projected population growth and the design flow rates is included in Appendix 2.

4.2.3 Water Demand

To calculate future water demand estimates, the unit demand values were multiplied by the projected number of future users in the water distribution system. Using the geometric method, the water demand estimates were calculated based on parameters in Table 4.2.

Table 4.4 : Details of population and rate of demand

Year	Population	AADD (m^3/day) including losses and peak factor	Demand per house (l/s)
2015	534	192.2	0.0250
2020	607	218.5	0.0284
2025	689	248.0	0.0323
2030	783	281.9	0.0367
2035	889	320.0	0.0416

For the base year 2015, the demand per household with 6 people is 0.0250 litres per second and for the design year is 0.0416 litres per second based on the population in the design year. Using this rate of demand for the design year, the network must be designed for the obtained flow. The derivation of the demand per house for all the years is shown in Appendix 2.

4.3 Methodology

4.3.1 General

The hydraulic performance assessment of the existing network was the first step in investigating the WRN's adequacy in performing its function. Simulation and optimization of the upgraded network, which included new connections were performed after hydraulic performance was satisfied and improvements on the existing system were proposed.

4.3.2 Approach

The methodology followed in this study involved assessing the hydraulic performance of the existing water distribution network and proposing upgrades to the network so that water is supplied to end users at appropriate pressure and flow to meet the demand. The focus was on the water flow from the tanks to the end users, pressure at different points within the distribution system, pipe materials and age of the distribution system. The input data comprised pipe type, lengths, sizes and roughness coefficients; elevation at the junctions and nodal demands based on population data of the area.

Future water demands for the Village were computed from the population data in Table 4.3 and the per capita demand as per the local guidelines (DHS 2019). This was followed by creating a hydraulic model of the network for use within EPANET. The as-built drawings of the existing WDN were used to prepare the backdrop for the

EPANET network. After creating the model, the network was then analysed using EPANET, and the results checked against the design criteria (DHS 2019). A model of the upgraded WDN that included houses not currently connected to the network was then developed and simulated. Trial and error optimization was undertaken on the upgraded WDN and thereafter optimization was performed with the Wadiso software.

Rossman *et al.* (2020) recommend the following steps when using EPANET to model a WDS.

- Drawing a network representation of the distribution system. This can be accomplished by importing a basic description of the network placed in a text file.
- Editing the properties of the objects that make up the system.
- Describing how the system is operated.
- Selecting a set of analysis options.
- Running a hydraulic/water quality analysis.
- Viewing the results of the analysis

4.4 Hydraulic performance assessment of the existing WDN

The hydraulic performance of the existing WDN comprising of 54 yard connections and three tanks, was undertaken as the first part of this study. The EPANET model of the existing network is shown in Figure 4.9.

EPANET software was used to compute the nodal pressures, pipe flows and pipe velocities during the analysis. These output parameters were used to identify areas of higher and lower pressure zones. The results of the EPANET analysis are presented in subsection 5.1 and were compared to the assessment criteria in Table 4.5 (DHS 2019).

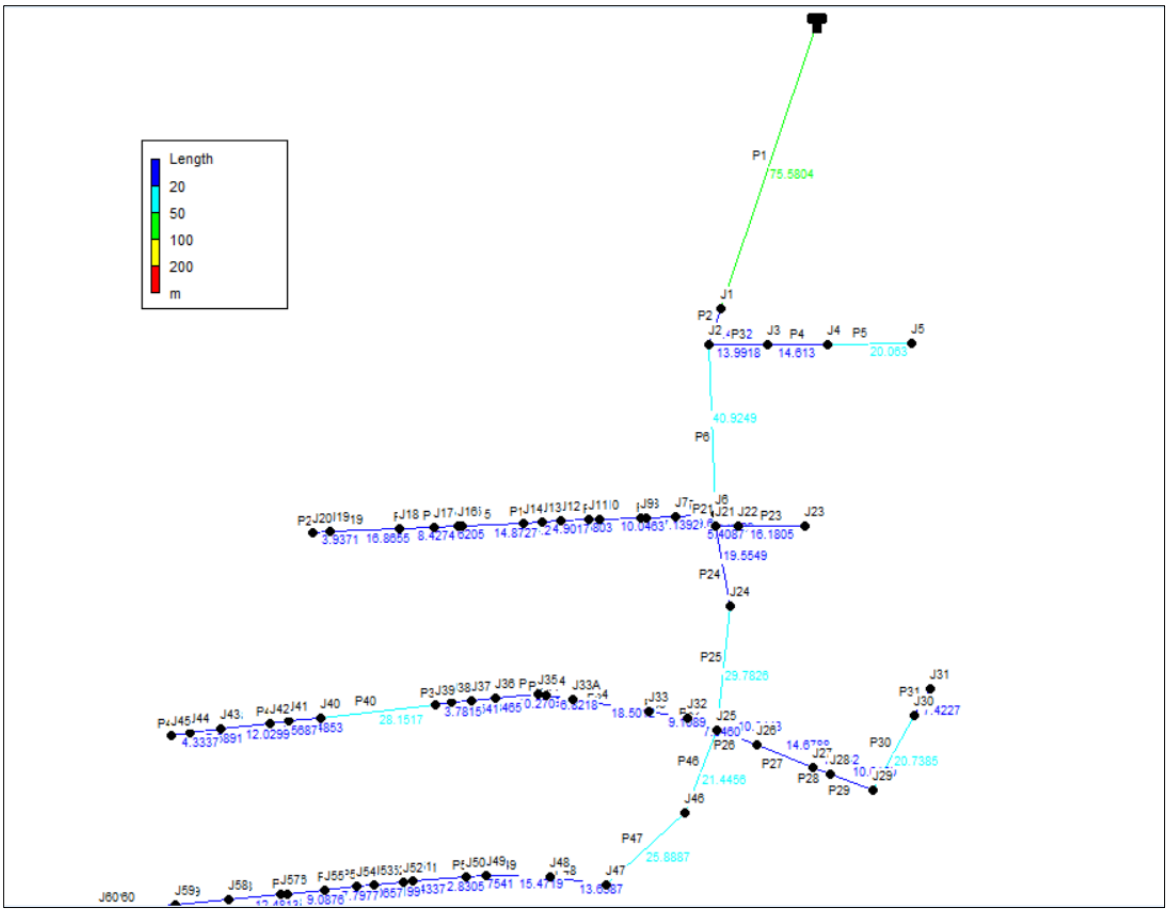


Figure 4.9 : EPANET model of existing Tswelopele Village WDN

Table 4.5 : Existing WDN assessment criteria (DHS 2019, CSIR 2000)

Item	Criterion	Value
1	Minimum Pressure	20m (*10m/ **15m)
2	Maximum Pressure	90m
3	Minimum Velocity	0.6m/s
4	Maximum Velocity	1.2m/s
5	Minimum actual internal diameter (serving two stands)	20mm, branching to 2 x 15mm
6	Minimum actual internal diameter (serving one stand)	15mm
7	Per capita demand	80L/c/d

* For dwelling houses with yard taps and yard tanks, the minimum head during design peak flow should be 10m adjusted with height difference between the main and highest point on the erf (CSIR 2000).

** For high-lying areas surrounding a reservoir an absolute minimum pressure of 15m is acceptable (DHS 2019)

4.4.1 Skeletonization of the Network

For ease of modelling and analysis, the network was skeletonised by only including main reticulation pipes and nodes, excluding yard connections. The demands on the junctions that were excluded were distributed to the main nodes. The skeletonized network consisted of 1 tank (3 tanks combined), 16 nodes and 16 pipes of same material as shown in Figure 4.10.

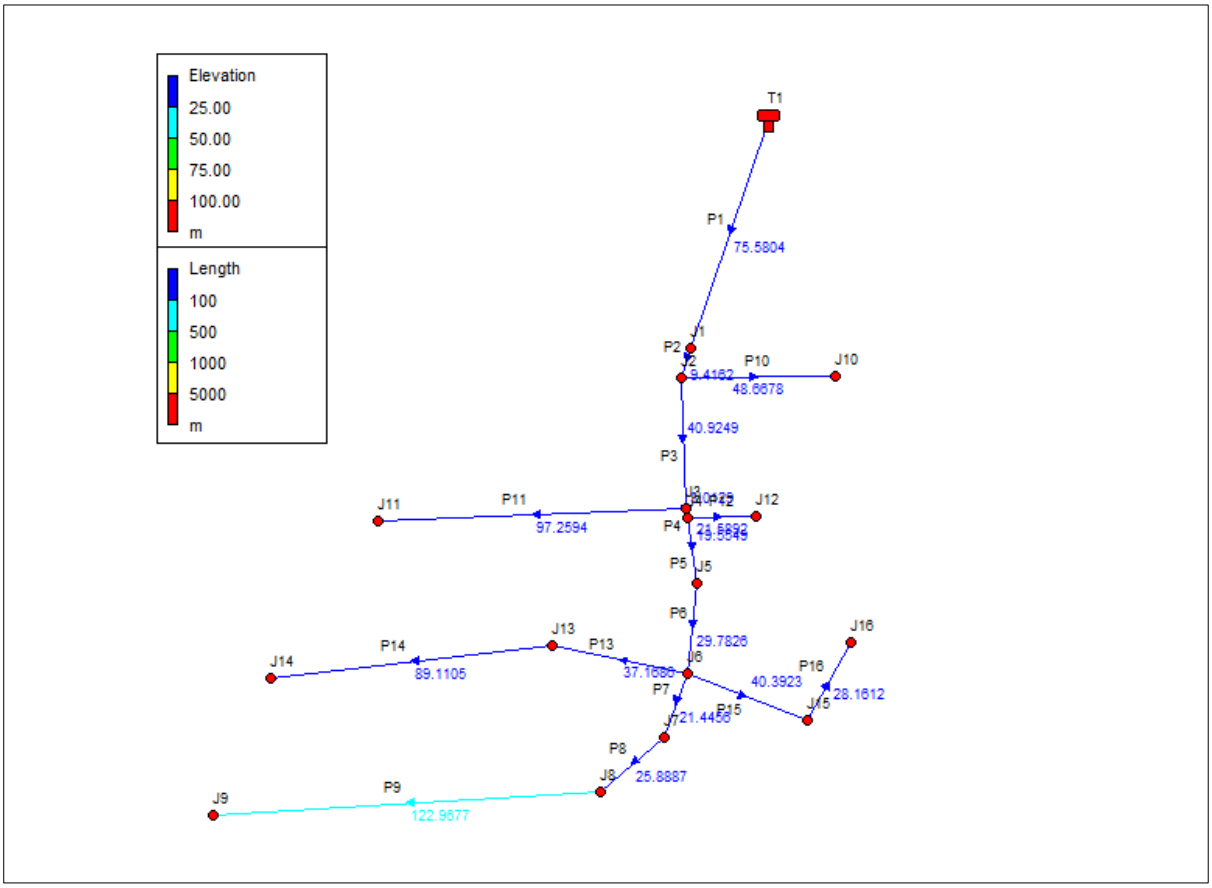


Figure 4.10 : Skeletonised EPANET model of existing Tswelopele Village WDN

4.4.2 Assigning distribution network parameters

After skeletonization of the network in EPANET, the network parameters were assigned to the links and nodes. The network parameters assigned to the network include nodal demands, node elevations, pipe lengths, pipe diameters and roughness coefficients. The existing network comprises 50mm diameter mainline and branch pipes, with a roughness of 130. The existing network pipe data is shown in Table 4.6.

Table 4.6 : Existing network pipe properties

Pipe ID	Start Node	End Node	Length (m)
P1	Tank	J1	75.58
P2	J1	J2	9.42
P3	J2	J3	40.92
P4	J3	J4	3.01
P5	J4	J5	19.55
P6	J5	J6	29.78
P7	J6	J7	21.45
P8	J7	J8	25.89
P9	J8	J9	122.97
P10	J2	J10	48.67
P11	J3	J11	97.26
P12	J4	J12	21.59
P13	J6	J13	37.17
P14	J13	J14	89.11
P15	J6	J15	40.39
P16	J15	J16	28.16

4.4.3 Nodal Demand Estimations

For the simulation of the existing network, the nodal demands were estimated by multiplying the per capita demand and the number of people in every household. This product was then multiplied with a factor for real losses (due to leakages) and peak factors to cater for reticulation losses and instantaneous peaks. It was then converted to litres per second for consistency with EPANET.

Nodal Demand = No. of people per household*demand*real losses factor*peak factor

= 6 people*80l/c/d*1.25*3.6

= 2160l/d

= 0.025l/s

Assumptions:

i. A 25% estimate was used for the real losses due to lack of data or knowledge of the Village (DHS 2019).

ii. A peak hour factor (PH_{hour}) of 3.6 for low-income housing (DHS 2019) was used.

4.4.4 Assigning node elevation data

The elevation data was extracted from google earth and assigned to all the nodes of the network. The elevations and nodal demands were entered into EPANET individually as per Table 4.7 shown below.

Table 4.7 : Existing network node properties

Node	Elevation (m)	Demand (l/s)
Tank	1405	0.000
J1	1388.0	0.025
J2	1385.0	0.000
J3	1379.3	0.000
J4	1378.7	0.000
J5	1375.9	0.025
J6	1372.6	0.000
J7	1369.9	0.000
J8	1367.5	0.000
J9	1363.2	0.325
J10	1384.6	0.075
J11	1376.8	0.350
J12	1378.7	0.050
J13	1372.1	0.100
J14	1368.9	0.275
J15	1371.9	0.075
J16	1374.6	0.050

4.5 Analysing the existing system with Wadiso

The initial step in analysing the existing system with Wadiso involves preparing the layout of the WDN from the background drawing. The input data for the nodes and pipes such as elevations, demands, lengths, diameter, roughness coefficients and tank parameters are then entered. Same input parameters used in the EPANET steady state simulation of the upgraded network can be used for this exercise. After capturing and saving the input data, the existing system is analysed. This process is referred to as balancing the system (GLS 2023), whereby the system calculates the flows in the pipes and pressures at the nodes. Since Wadiso runs on EPANET engine, an EPANET status report is issued after the steady state simulation.

4.6 Simulation of upgraded WDN

To accommodate additional connections an upgraded network was proposed which included additional thirty-five households that were not initially connected. The initial design incorporated the existing 50mm diameter pipes without looking at replacing them to save costs. The existing WDN layout was adopted, and the new houses added. The main objective of developing a WDN is to cater for future demands after a certain design period. For this study, a design period of 20 years was selected based on the DHS (2019) guidelines. The water demand data in section 4.2.4 was used for the nodal demands of the upgraded WDN.

4.6.1 Assigning invert elevation data to additional nodes

The elevation and demand data including data for the new nodes (serving additional 35 houses) of the upgraded WDN are shown in Table 4.8. The demand at each node was assumed by assigning an equal future demand per each yard and utilizing the skeletonized network.

Table 4.8 : Junction details of network (with updated nodal demands)

Node	Elevation (m)	Demand (l/s)
Tank	1405	N/A
J1	1388	0.000
J2	1385.0	0.000
J3	1379.3	0.000
J4	1378.7	0.000
J5	1375.9	0.042
J6	1372.6	0.000
J7	1369.9	0.000
J8	1367.5	0.000
J9	1363.2	0.583
J10	1384.6	0.125
J11	1376.8	0.624
J12	1378.7	0.083
J13	1372.1	0.166
J14	1368.9	0.624
J15	1371.9	0.125
J16	1374.6	0.083
J17	1367.3	0.208
J18	1372.2	0.000
J19	1372.7	0.000
J20	1366.4	0.416
J21	1374.1	0.000
J22	1371.2	0.333
J23	1389.7	0.291

4.6.2 Assigning distribution network parameters

The input parameters of the extra pipes (serving additional 35 houses) for the upgraded WDN are shown in Table 4.9. The table shows the start and end node for each pipe and length of the pipe. The proposed diameter and roughness of all the proposed pipes, are 50mm and 130 respectively.

Table 4.9 : Length of the additional pipes of the upgraded WDN

Pipe ID	Start Node	End Node	Length (m)
P17	J15	J17	44.04
P18	J15	J18	8.81
P19	J18	J19	20.23
P20	J19	J20	43.35
P21	J19	J21	30.11
P22	J21	J22	30.39
P23	J1	J23	65.4

After inputting these parameters, the analysis of the WDN was performed with EPANET and the performance of the WDN checked against the design criteria. This process was terminated when the smallest possible diameter pipes, which are cheap were determined.

4.7 Optimization of upgraded WDN using trial and error

The objective of the trial and error procedure is to identify the least expensive network design that meets the design criteria i.e., pressure and velocity requirements as per the guidelines. Therefore, the process described in section 4.6 was repeated until a better solution was realised based on engineering judgement. The results of the trial and error procedure are included in section 5.2.

4.8 Optimization of upgraded WDN using Wadiso optimization software.

After determining a preferred solution with trial and error optimization, Wadiso was then employed to analyse and optimize the upgraded Tswelopele Village WDN. The technique employed by Wadiso is simple enumeration where the user specifies candidate sizes for each group of pipes to be sized, and the demands and minimum pressures that the network is to comply with (GLS 2023). This procedure does not only rely on engineering judgement or experience like the trial and error optimization. The optimization process was undertaken by utilizing the existing pipes and new pipes for the additional houses.

4.8.1 Analysing and optimizing the upgraded network

After building the model of the existing system, the network was prepared for optimization of the future system. This was completed by extending the existing

system to include the additional 35 households. Network input parameters for the upgraded network were captured and saved.

i. Defining the cost data

The initial step of the optimization process is defining the cost data (GLS 2023). In this study the pipe cost is the only cost data entered. The cleaning and lining of pipes, tank and pump costs could also be entered if they are part of the optimization problem. The same pipe costs used for the trial and error optimization were also used in Wadiso optimization.

Table 4.10 : Pipe cost data

Diameter (mm)	**Cost per metre (R/m)**
20	R20
32	R30
40	R50
50	R65
63	R100
75	R143
90	R204

ii. Grouping of pipes and assignment of cost data

For the purposes of optimization with Wadiso software, it is important that pipes to be optimized are grouped as it allows the designer to dictate the design. The pipes to be optimized were selected and assigned to groups based on preferred diameters. The pipes with similar diameter were placed in the same group. Four pipe groups were proposed as shown in Table 4.11.

Table 4.11 : Upgraded WDN pipe groups.

Pipe Group	**Pipes in Group**	**Candidate Sizes (mm)**	**Cost Function**
1	1, 2, 3, 4, 5, 6	50, 63, 75	FN1
2	7, 8, 9, 11, 13, 14, 15	40, 50, 63	FN1
3	10, 12, 16	32, 40, 50	FN1
4	17, 18, 19, 20, 21 22, 23	40, 50, 63	FN1

After grouping the pipes to be optimized, cost functions for those pipes were assigned using engineering judgement. Cost functions of pipes of similar diameters may differ due to other conditions such as the excavation methods to be used or the geotechnical conditions of the ground where the pipe will be placed. This can also depend on whether a single or double pipeline is to be constructed in the same trench.

iii. Defining candidate sizes for groups

Wadiso allows up to 10 different pipe sizes to be given for each pipe group. Cleaning and relining of the existing pipes can be achieved by assigning “C” value as a pipe size to the relevant pipe group. Pipe sizes ranging from 32mm to 75mm were selected as the possible candidate sizes for the groups. They were assigned to the four pipe groups as shown in Table 4.11.

iv. Defining the design loads/outputs

Finally, before running the optimization, the loads (nodal demands) and the pressure requirements which define the boundary conditions of the optimization problem, were entered. For this study, the loads were taken as the nodal demands and the minimum pressure of 20m was entered as the boundary condition. The upgraded network and the input data for Wadiso is shown in Appendix from item (i) to (v).

v. Running the optimization

After completion of the above steps, Pareto tolerances settings were applied to the model before optimization. The Pareto tolerances were set to 5m for pressure and 10% for costs. Thereafter, the optimization of the network was performed with the enumeration process testing all the possible combinations by applying a cost test and a size test, and thereby eliminating infeasible solutions. The optimization results are presented and discussed in section 5.3 of this report and also shown in Appendix 1 item (vi) to (xi).

5. RESULTS AND ANALYSIS OF FINDINGS

5.1 EPANET analysis results of the existing WDN

The pressure at different nodes, velocities and flows for the pipes were noted after simulation with EPANET and are discussed below.

i. Variation of pressure for different nodes for the existing WRN

After simulation of the model with EPANET, the pressure values at all the nodes were checked and they were found to be above the minimum required except for node J1 with a pressure head of 17.97m. As per the DHS (2019), pressure should be between 20m and 90m, but this can be relaxed to absolute minimum of 15m. The pressure results at the nodes are shown in Figure 5.1 and Table 5.1. These results indicate that all the nodes have sufficient pressure, and all households are supposed to receive water at optimal conditions. However, these results were not verified using field measurements and observations because of logistical constraints.

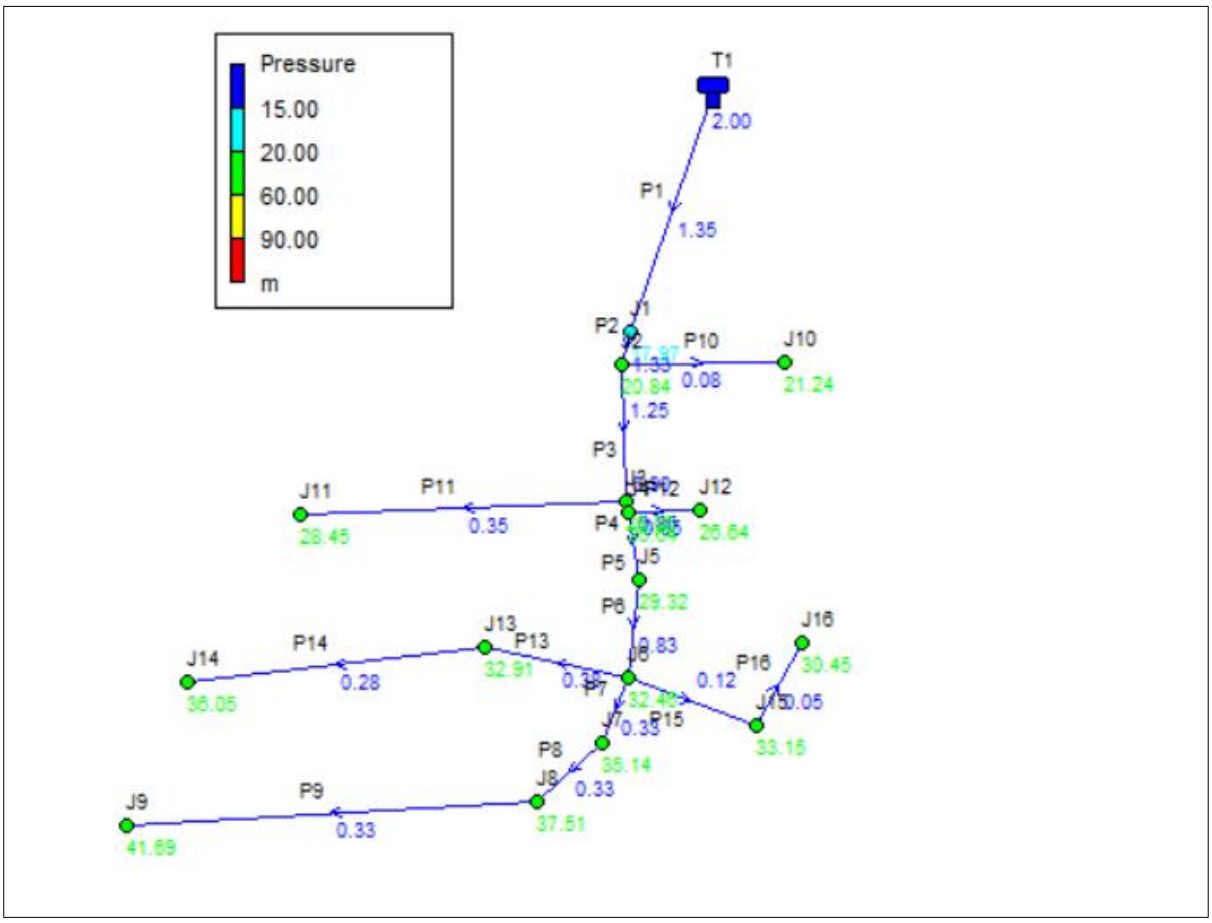


Figure 5.1 : Pressure and flow distribution network diagram of the existing WDN

Table 5.1 : Pressure results for existing WDN

Node ID	Demand (l/s)	Pressure (m)
Tank	-1.35	2.0
J1	0.03	17.97
J2	0.00	20.84
J3	0.00	26.06
J4	0.00	26.64
J5	0.03	29.32
J6	0.00	32.46
J7	0.00	35.14
J8	0.00	37.51
J9	0.33	41.69
J10	0.08	21.24
J11	0.35	28.45
J12	0.05	26.64
J13	0.10	32.91
J14	0.28	36.05
J15	0.08	33.15
J16	0.05	30.45

ii. Variation of flow for different links in the existing WRN

The pipes with highest flows are P1, P2 and P3 with flows of 1.35, 1.33 and 1.25 litres per second respectively. The lowest flows are in pipes P10, P12 and P16 with flows of 0.08, 0.05 and 0.05 litres per second respectively.

iii. Variation of velocity for different links in the existing WRN

Simulation results of velocity obtained from the EPANET analysis are presented in Figure 5.2 and Table 5.2. Velocities through pipes P1, P2 and P3 are within the limits and the rest are below 0.6m/s. The pipes with higher velocities reflect their proximity to the source of water.

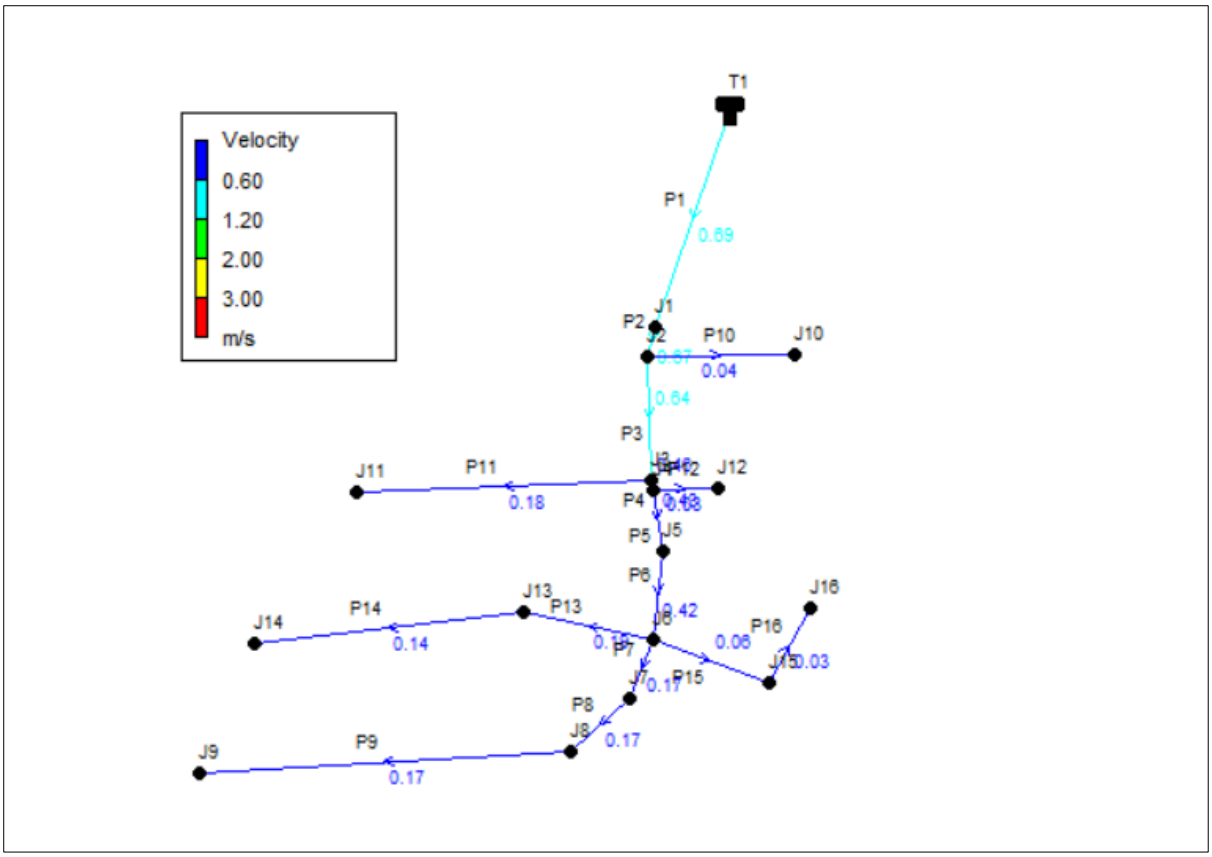


Figure 5.2 : Velocity results of the existing WDN

Table 5.2 : Link output parameters for the existing WDN

Pipe ID	Velocity (m/s)	Flow (lps)
P1	0.69	1.35
P2	0.67	1.33
P3	0.64	1.25
P4	0.46	0.90
P5	0.43	0.85
P6	0.42	0.83
P7	0.17	0.33
P8	0.17	0.33
P9	0.17	0.33
P10	0.04	0.08
P11	0.18	0.35
P12	0.03	0.05
P13	0.19	0.38
P14	0.14	0.28
P15	0.06	0.08
P16	0.03	0.03

Overall, the results obtained confirmed that the pressures at all junctions are viable to provide adequate water to the Tswelopele network. However, the flow and velocity results indicate that there might be challenges in supplying water to households extracting water from all the nodes after J2 because the velocities in pipelines P4 to P16 is less than 0.6m/s. One possible explanation for this discrepancy between the model results and what is obtaining on the ground where water supply challenges have been reported at households supplied by pipe P9, P10, P15 and P16, might be the presence of leakages, half open valves or illegal connections in the network.

5.2 Optimization results of the upgraded WDN using trial and error

The upgraded WDN was simulated using the EPANET software, and the pressure and velocity results were checked against the design criteria. Where the results were not within the permissible limits as per the guidelines, the diameter of certain pipes within the network were adjusted and the WDN simulated again using EPANET. This trial and error process was repeated until output parameters, such as pressure and velocity, were within or near the permissible ranges. The steps taken for the trial and error optimization process are given in Table 5.3.

The initial trial utilised 50mm diameter pipes for both the mainline and the branches and the EPANET results were unsatisfactory because several nodes had pressure below 20m and the velocity in most pipes was below 0.6m/s. Therefore, other trials with different combinations of pipe sizes ranging from 40mm to 90mm, were undertaken in order to improve the design. The final trial involved varying individual pipes whilst maintaining the same network from the tanks to the yards. The final network which produced acceptable pressure at most of the nodes comprised of pipes ranging from 40mm to 75mm diameter.

Table 5.3 : Trial and error results for the upgraded network.

Trial	Variables	Results
1	50mm diameter mainline & 50mm diameter branches	The pressure in nodes J1, J2, J10 and J23 is below 20m. The pipes with velocities within the acceptable range of 0.6m/s to 1.2m/s are P1, P2, P3, P4, P5 and P6. The velocity within the other pipes is below 0.6m/s. **Remedial Action:** Increase the pipe diameter of P1, P2, P3, P4, P5 and P6 to 63mm and check the parameters again.
2	63mm diameter mainline & 50mm diameter branches	The pressure in node J23 is 18.39m which is below the minimum accepted pressure. The pipes with velocities within the acceptable range of 0.6m/s to 1.2m/s are P1, P2, P3, P4, P5 and P6. The velocity within the other pipes is below 0.6m/s. **Remedial Action**: Increase the pipe diameter of P1, P2, P3, P4, P5 and P6 to 75mm and check the parameters again.
3	75mm diameter mainline & 50mm diameter branches	The pressure in node J23 is 19.63m which is below the minimum accepted pressure. The pipes with velocities within the acceptable range of 0.6m/s to 1.2m/s are P1, P2, P3 and P4. The velocity within the other pipes is below 0.6m/s. **Remedial Action**: Increase the pipe diameter of P1, P2, P3, P4, P5 and P6 to 90mm and check the parameters again.
4	90mm diameter mainline & 50mm diameter branches	The pressure in all the nodes is above 20m. Velocity in all pipes is less than 0.6m/s. **Remedial Action**: Try smaller mainline diameter pipe and check the parameters again
5	63mm diameter mainline & 63mm diameter branches	The pressure in node J23 is 18.42m which is below the minimum accepted. The pipes with velocities within the acceptable range of 0.6m/s to 1.2m/s are P1, P2, P3, P4, P5 and P6. The velocity within the other pipes is below 0.6m/s. **Remedial Action**: Increase the pipe diameter of P1, P2, P3, P4, P5 and P6 to 75mm and keep the branches at 63m and check the parameters again.
6	75mm diameter mainline & 63mm diameter branches	The pressure in node J23 is 19.66m and is below minimum accepted.

		The pipes with velocities within the acceptable range of 0.6m/s to 1.2m/s are P1, P2, P3 and P4. The velocity within the other pipes is below 0.6m/s. **Remedial Action**: Decrease the pipe diameter of P1, P2, P3, P4, P5 and P6 to 63mm and decrease the pipe diameter of the branches to 40m.
7	63mm diameter mainline & 40mm diameter branches	The pressure in node J23 is 18.28m which is below the minimum accepted pressure. The pipes with velocities within the acceptable range of 0.6m/s to 1.2m/s are P1, P2, P3, P4, P5, P6, P13 and P15. The velocity within the other pipes is below 0.6m/s. **Remedial Action**: Increase the pipe diameter of P1, P2, P3, P4, P5 and P6 to 75mm and keep the pipe diameter of the branches at 40m.
8	75mm diameter mainline & 40mm diameter branches	The pressure in node J23 is 19.53m and is below minimum accepted. The pipes with velocities within the acceptable range of 0.6m/s to 1.2m/s are P13 and P15. The velocity within pipes P1, P2, P3, P4, P5 and P6 is above 0.6m/s and velocity within all the other pipes is below 0.6m/s. **Remedial Action**: Try pipes with smaller diameter for the mainline.
9	50mm diameter mainline & 40mm diameter branches	The pressure in nodes J1, J2, J10 and J23 is below 20m. The pipes with velocities within the acceptable range of 0.6m/s to 1.2m/s are P13 and P15. The velocity within pipes P1, P2, P3, P4, P5 and P6 is above 1.2m/s and velocity within all the other pipes is below 0.6m/s. **Remedial Action**: Based on the previous trials select the most efficient pipes using engineering judgement.
10	Final mixed pipe diameters for mainline and branches. **63mm:** P1, P2, P3, P4, P5, P6, P7, P8 **50mm**: P13, P15 **40mm**: P9, P10, P11, P12, P14, P16, P17, P18, P19, P20, P21, P22, P23	The pressure in node J23 is 18.28m and is below minimum accepted but above 15m which is the absolute minimum under certain conditions. The pipes with velocities within the acceptable range of 0.6m/s to 1.2m/s are P1, P2, P3, P4, P5, P6, P18 and P19. The velocity within all the other pipes is below 0.6m/s. **Action**: Accept this design as the final proposal based on trial and error.

5.2.1 Pressure results for the upgraded network (EPANET)

The final upgrade design comprising of mixed pipes ranging from 40mm to 63mm met the pressure requirements at all nodes except node J23 with a pressure of 18.28 (less than 3m from the minimum accepted). As per the DHS (2019), the acceptable pressure is supposed to be between 20m to 90m but pressure values below 20m but above 15m are acceptable under certain conditions. The pressure results at the nodes are shown in Figure 5.3 and Table 5.4. These results were accepted as the final results for trial and error even though some pipes were violating the velocity requirements.

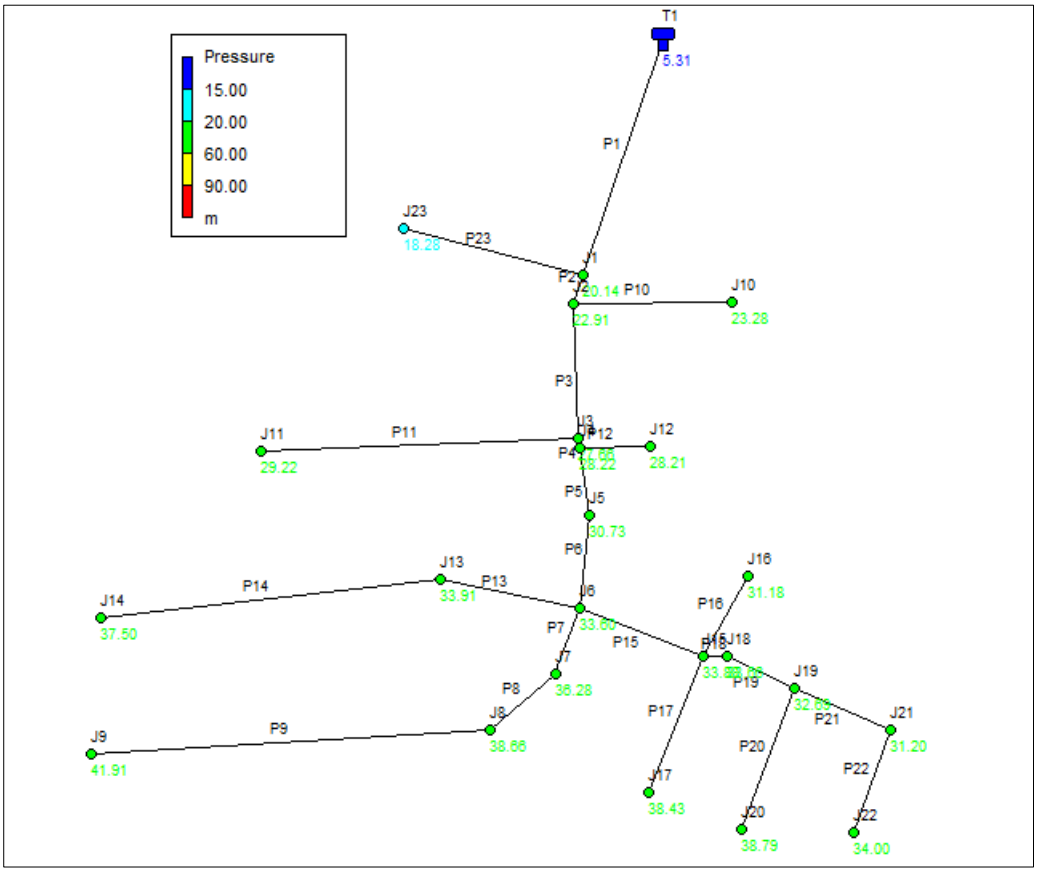


Figure 5.3 : Pressure distribution network diagram for the upgraded WDN

Table 5.4 : Pressure results at nodes: upgraded network (trial and error)

Node ID	Pressure (m)	Node ID	Pressure (m)
Tank	5.31	J12	28.21
J1	20.14	J13	33.91
J2	22.91	J14	37.50
J3	27.66	J15	33.88
J4	28.22	J16	31.18
J5	30.73	J17	38.43
J6	33.60	J18	33.66
J7	36.28	J19	32.69
J8	38.66	J20	38.79
J9	41.91	J21	31.20
J10	23.28	J22	34.00
J11	29.22	J23	18.28

5.2.2 Velocity results of the upgraded network (EPANET)

The EPANET simulation results of velocity of the upgraded network are shown in Figure 5.4 and Table 5.5.

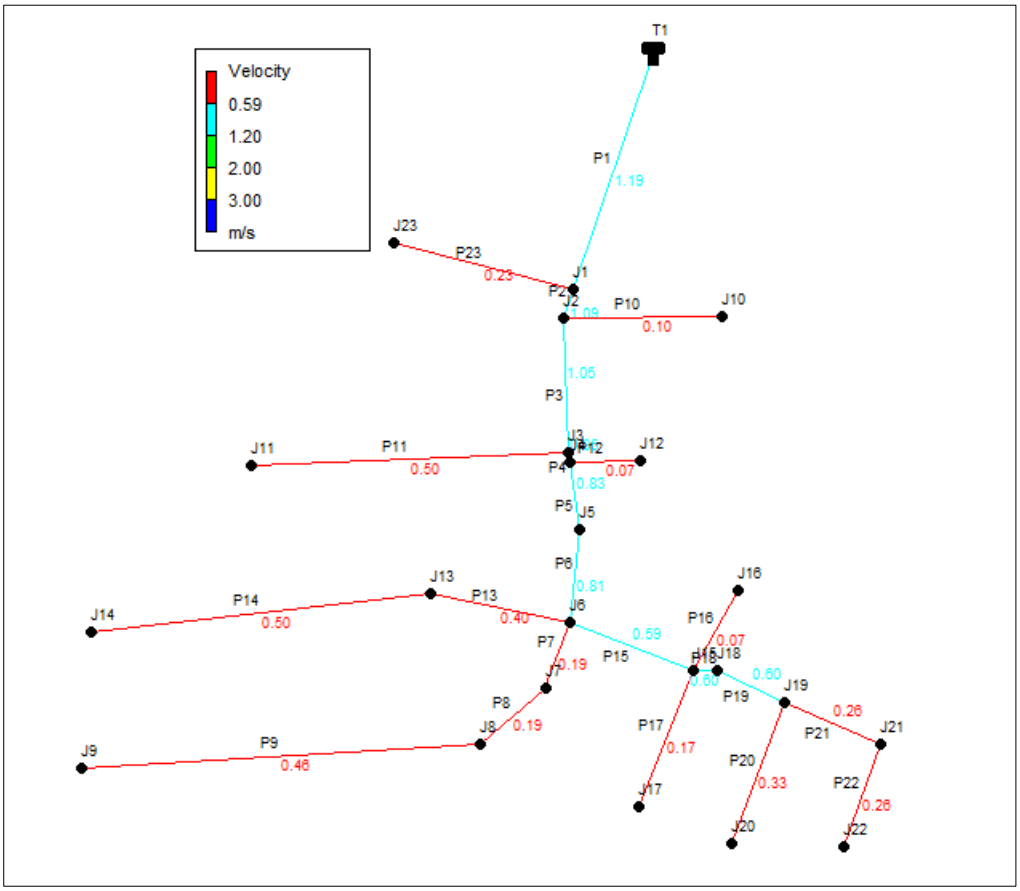


Figure 5.4 : Upgraded WDN pipe velocities

The velocity through P1, P2, P3, P4, P5, P6, P18 and P19 is within the acceptable limits. However, the velocity in all the other pipes is below the minimum required of 0.6m/s. Acceptable velocity in pipes can be achieved by decreasing the diameters of the affected pipes but the diameters in the affected pipes cannot be reduced any further as they are the minimum acceptable diameters which were producing results within the acceptable limits as per the design criteria. Scenarios 1 to 9 of the trial and error optimization were eliminated as possible solutions because the velocity in most of the pipes was not within the required limits.

Table 5.5 : Upgraded WDN links output parameters

Pipe ID	Diameter (mm)	Velocity (m/s)	Flow (lps)
P1	63	1.19	3.7
P2	63	1.09	3.41
P3	63	1.05	3.29
P4	63	0.85	2.66
P5	63	0.83	2.58
P6	63	0.81	2.54
P7	63	0.19	0.58
P8	63	0.19	0.58
P9	40	0.46	0.58
P10	40	0.1	0.12
P11	40	0.5	0.62
P12	40	0.07	0.08
P13	50	0.4	0.79
P14	40	0.5	0.62
P15	50	0.59	1.17
P16	40	0.07	0.08
P17	40	0.17	0.21
P18	40	0.60	0.75
P19	40	0.60	0.75
P20	40	0.33	0.42
P21	40	0.26	0.33
P22	40	0.26	0.33
P23	40	0.23	0.29

The outcome of this EPANET analysis was taken as the final design from the trial and error optimization for comparison with the design by Wadiso optimization.

5.2.3 Cost Estimate of the upgraded WDN based on trial and error

The cost estimate of the pipework for the upgraded network based on trial and error is R60,076.90 shown in Table 5.6 below. These cost estimates are based on the July 2023 rates obtained from HDPE Plumbing & Trading which were also used in Wadiso optimization.

Table 5.6 : Cost estimate of upgraded network (Trial and error)

Pipe diameter (mm)	**Length (m)**	**Cost per unit length**	**Amount (R)**
40	650.09	R50.00	R32 504.50
50	77.56	R65.00	R5 041.40
63	225.31	R100.00	R22 531.00
TOTAL			**R60,076.90**

5.3 Optimization results of the upgraded WDN using Wadiso

Results obtained by optimization with Wadiso software were instantly given in a tabular form just after running the optimization as shown in Table 5.7. According to GLS (2023), the process of enumeration tests each combination by applying a cost test and a size test thereby eliminating solutions that might not be feasible. The results were given after analysing 37 combinations and the Pareto Optimal results were included as alternatives. Pareto Optimal solutions are results produced by relaxing the conditions to produce sets of good results instead of just one solution. For this study, the pressure tolerance was set as 5m, meaning that a solution that produces nodal pressure of 15m and above was acceptable since the absolute minimum pressure for low income households with yard connections can be relaxed to 15m (DHS 2019).

Table 5.7 : Wadiso optimization results of the upgraded WDN

Alternative ID	**Total Cost (R)**	**Min Press violation**	**Group 1 Size**	**Group 2 Size**	**Group 3 Size**	**Group 4 Size**
Optimum	54,611.90	3.24	63	40	32	40
Alternative 1	48,372.50	-1.76	50	40	32	40
Alternative 2	50,340.90	-1.71	50	40	40	40
Alternative 3	51,817.20	-1.70	50	40	50	40
Alternative 4	56,580.20	3.29	63	40	40	40
Alternative 5	58,056.50	3.31	63	40	50	40

The optimum solution is shown in the second row of Table 5.7 with a total cost of R54,611.90. The pressure violation for this solution is 3.24m above the minimum specified of 20m. The actual sizes of different groups are also given. Furthermore, three Pareto optimal alternatives are also shown with cheaper total cost than the optimum but slightly violating the pressure requirements. The other two solutions are more expensive than the optimum but give better pressure. These results can be presented as shown in the graph depicted in Figure 5.5.

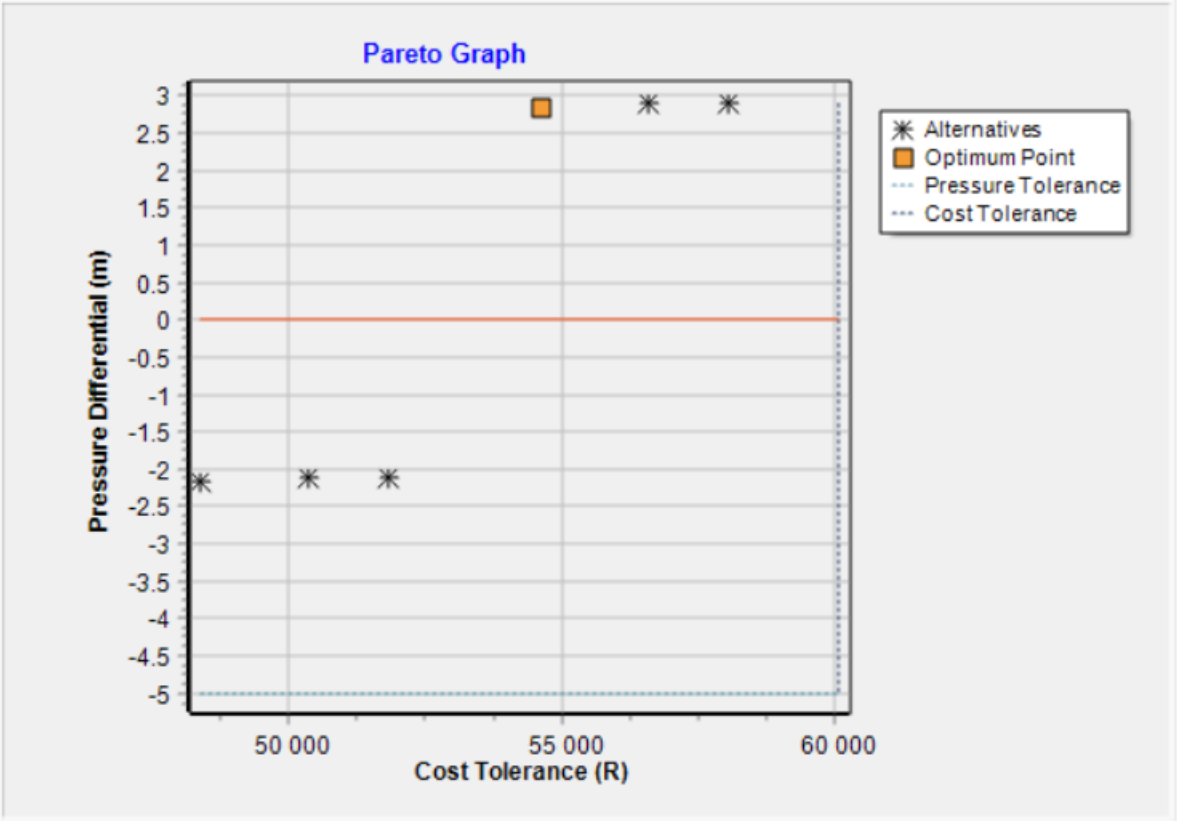


Figure 5.5 : Optimization results: Pressure differential vs cost

The Pareto Optimal theory is based on the idea that there might be another functional combination with a higher cost than the previous best solution within the pressure tolerance limits. Hence, the idea of considering Pareto Optimal solutions is to determine the optimal solution in terms of cost and minimum pressure (GLS 2023).

Similarly, a potential solution may pass the pressure test even if the pressures are below the agreed minimum as per standards and/ or guidelines, but still within the specified tolerances. Therefore, some solutions which slightly fail the pressure test are included in the Pareto Optimal set. The advantage of including the Pareto Optimal combinations is that it gives helpful information to designers and also presents some combinations which might have been excluded because of strict conditions. So, with Pareto Optimum, systems with better performance can be generated by allowing minor additional expenditure and also systems with pressures slightly lower than the specified requirement can also be considered. Since WDNs are designed for a certain design period, it might be possible that the solution with pressures slightly less than the requirement, will only fail for less than 20% of the design period when the population has significantly increased. For this exercise,

‘alternative 1’ solution shown in Table 5.7 is selected since it is the least expensive with a total cost of $48,372.50.

Results for ‘alternative 1’ solution are given in Table 5.8.

Table 5.8 : Wadiso optimization results for the upgraded WDN

Pipe ID	Optimized diameter (mm)	Velocity (m/s)	Flow (lps)
1	50	1.89	3.70
2	50	1.74	3.41
3	50	1.67	3.29
4	50	1.36	2.66
5	50	1.31	2.58
6	50	1.29	2.54
7	40	0.46	0.58
8	40	0.46	0.58
9	40	0.46	0.58
10	32	0.16	0.12
11	40	0.50	0.62
12	32	0.10	0.08
13	40	0.63	0.79
14	40	0.50	0.62
15	40	0.93	1.17
16	32	0.10	0.08
17	40	0.17	0.21
18	40	0.60	0.75
19	40	0.60	0.75
20	40	0.33	0.42
21	40	0.26	0.33
22	40	0.26	0.33
23	40	0.23	0.29

Table 5.9 : Pressure results at nodes: upgraded network (Wadiso)

Node ID	Pressure (m)	Node ID	Pressure (m)
Tank	N/A	12	21.14
1	15.63	13	25.00
2	17.91	14	24.14
3	20.70	15	24.51
4	21.16	16	24.50
5	23.07	17	25.59
6	25.06	18	24.09
7	27.58	19	23.32
8	27.77	20	24.26
9	28.70	21	21.83
10	17.84	22	23.01
11	19.76	23	19.04

5.4 Comparison of costs (trial and error vs Wadiso optimization)

The 10th iteration of the trial and error optimization was accepted as the preferred network for that process even though it could not be ascertained if it was the optimal solution. It is difficult to determine the optimal solution using trial and error hence the least-cost design from those obtained by trial and error which meets the design criteria is regarded as the best solution.

The solution by Wadiso violates the pressure requirements of 20m but it is still above the absolute minimum pressure (DHS 2019). This final solution falls within the Pareto tolerances and is accepted as a better solution than the one produced by trial and error since its cheaper. When comparing the velocities of the solutions obtained by the two approaches, the solution produced by Wadiso produced higher velocity values.

Optimization by trial and error method produced an upgraded WDN design costing R60,076.90 whilst the optimization by Wadiso produced an upgraded design costing R48,372.50. The cost comparison of the solutions indicates that a saving of 19.5% was realised by using Wadiso optimization software.

6. CONCLUSION AND RECOMMENDATIONS

As per the objectives of the research the existing Tswelopele WDN was modelled in EPANET, and all the required parameters were included for analysis. Initially the hydraulic performance of the existing WDN was determined and then upgrades were proposed to the network. The upgraded network was analysed and optimized initially with trial and error and also with Wadiso. The cost of the pipes obtained from both approached were then compared.

A summary of the key highlights of the objectives is presented.

i. Objective 1: To use an EPANET model to simulate the existing Tswelopele WDN and find (and/or verify) the nodes experiencing below minimum pressures and flows.

The model of the existing network prepared was analysed in EPANET and the results indicated that pressure at one junction was below the minimum required pressure of 20m. The velocity within more than 80% of the pipes was below the minimum required velocity of 0.6m/s. The results obtained showed that the pressure at all junctions and the flows at all pipes are large enough to provide adequate water to the network of the study area. Even though there was only one node that was identified with pressure marginally below minimum accepted (18m), it could not be ascertained why there were some households which were experiencing low pressure. Some possible causes of low pressure in a WDN besides poor designs include leakages, partially open valves and illegal connections.

ii. Objective 2: To determine and incorporate into the Tswelopele WDN, the upgrades required to accommodate current and future water demands.

The proposed upgrade of the existing network included connecting additional 35 houses to Tswelopele WDN. Future water demands for the upgrade were calculated based on the projected 2035 population obtained using the 20 year design period.

iii. Objective 3: To optimize the proposed upgrade to the Tswelopele WDN using trial and error typically employed by industry.

Optimization by trial and error was conducted on the upgraded network. After iterating 10 times a solution was accepted which comprised 40mm, 50mm and 63mm diameter pipes. The pressure in all the nodes except one, were all above the minimum acceptable pressure. However, varying the diameter to achieve acceptable

velocities proved a challenge as this required much smaller pipes which were below the acceptable criteria set in DHS Red Book (DHS 2019).

iv. Objective 4: To optimize the proposed upgrade to the Tswelopele WDN using the Wadiso software.

Wadiso software was used to optimize the upgraded Tswelopele WDN. Starting with a network with 50mm diameter pipes the optimization process converged to an optimal solution with pipes ranging from 32mm for branch pipes to 63mm diameter pipes on the mainline. Pareto tolerances of 5m on minimum pressure and 10% cost were applied. An alternative to the optimum design was selected as the preferred solution since it had the least cost but still within the Pareto tolerance, which is acceptable.

v. Objective 5: To compare the results obtained using trial and error with those obtained using Wadiso.

Trial and error optimization and Wadiso optimization produced comparable results in terms of pressure at the nodes, but Wadiso produced results with better velocities. Comparison of cost estimates between the solutions produced by optimization by trial and error and those produced by Wadiso optimization shows a cost saving of approximately 19.5% by employing Wadiso.

In conclusion, the following recommendations are given:

i. It is recommended that a larger network and bigger population with higher demands could have clearly highlighted the difference in performance of these two approaches to optimization. However, even though with a smaller network like Tswelopele WDN, it has been shown that optimization of WDNs can help with reducing the costs of pipework.
ii. Further application of optimization techniques should be encouraged in Southern African region as this might help with constructing optimal networks which are cheaper, reliable, and efficient. Furthermore, multi-objective optimization must also be pursued in the design of WDNs in Southern Africa because of its incorporation of objectives other than cost in the optimization process.

iii. The challenge with most commercial software is that they are expensive and some consulting firms do not afford them. Therefore, there is a need for freely available software for optimizing WRN design..

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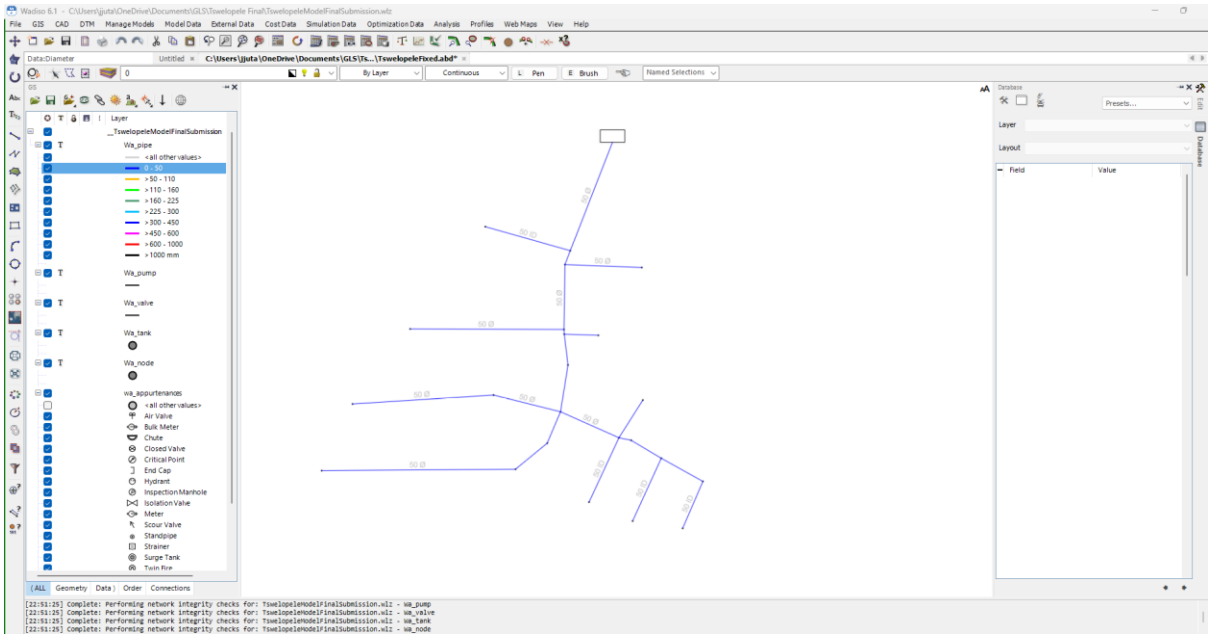
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APPENDIX 1: WADISO ANALYSIS & OPTIMIZATION SCREENSHOTS

i. Upgraded network layout.



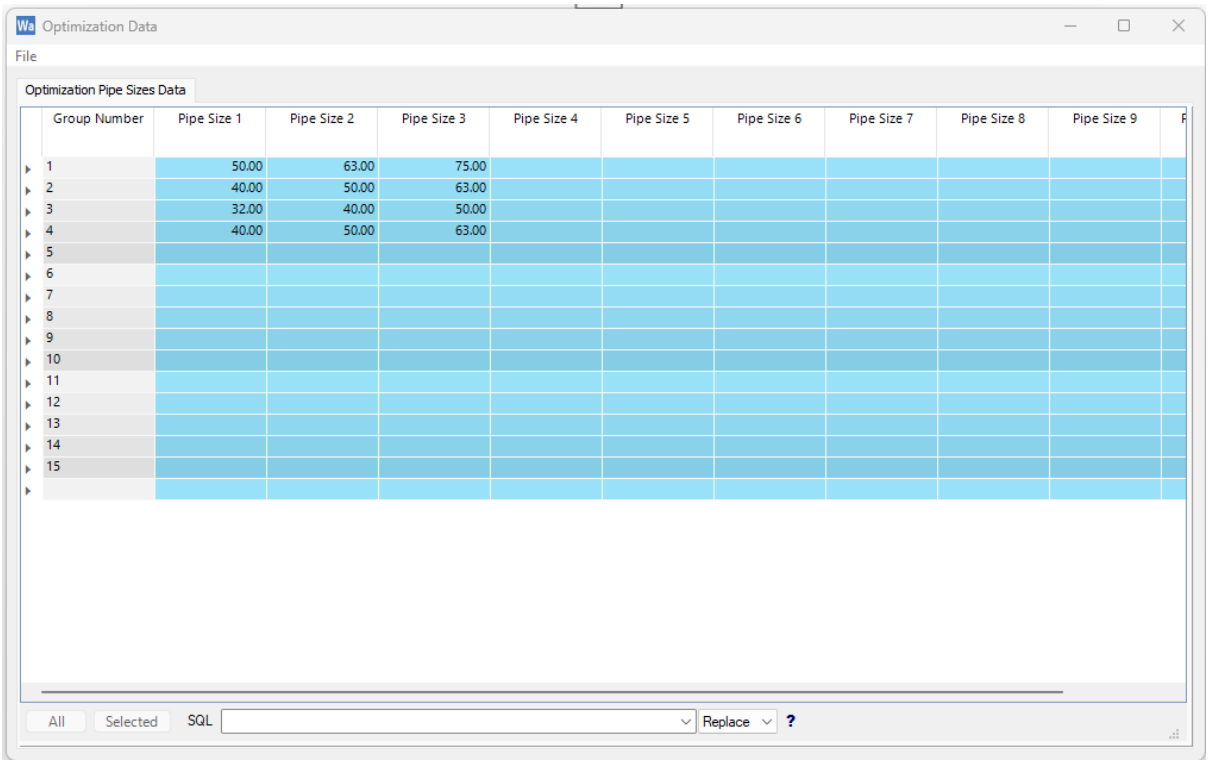
ii. Cost functions

Wadiso Cost File - Pipe Cost Data - C:\Users\jjuta\OneDrive\Documents\GLS\Tswelopele\Wadiso 6 - Tutorial Cost Data.sqlite

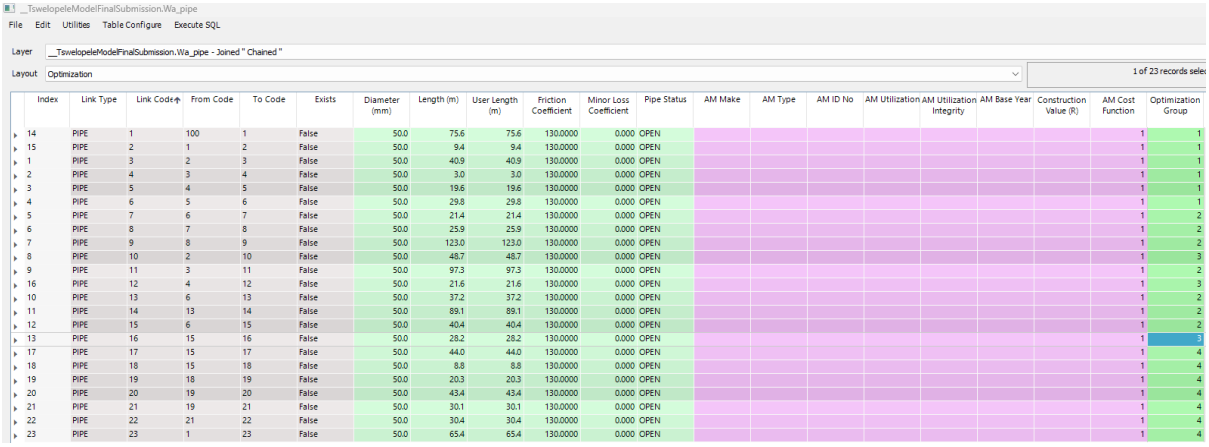
File Tables

Inside Diameter [mm]	Nominal Diameter [mm]	Cost1	Cost2	Cost3	Cost4	Cost5	Cost6	Co
20	20	20.00	26.00	30.00				
32	32	30.00	39.00	45.00				
40	40	50.00	65.00	75.00				
50	50	65.00	84.50	97.50				
63	63	100.00	130.00	150.00				
75	75	143.00	185.90	214.50				
90	90	204.00	265.30	306.00				

iii. Optimization pipe sizes data



iv. Pipe table including design groups and cost functions.



v. Node table

_TswelopeleModelFinalSubmission.Wa_node

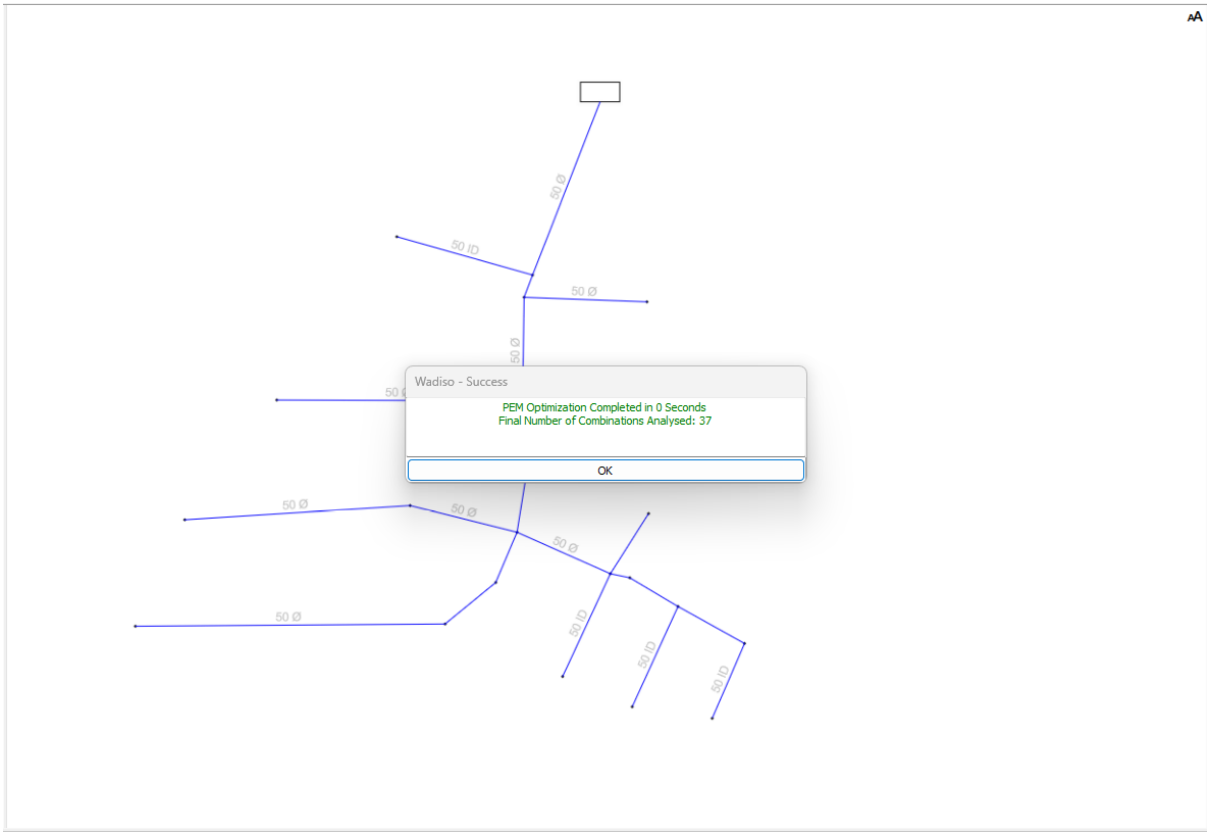
File Edit Utilities Table Configure Execute SQL

Layer _TswelopeleModelFinalSubmission.Wa_node - Joined " Chained "

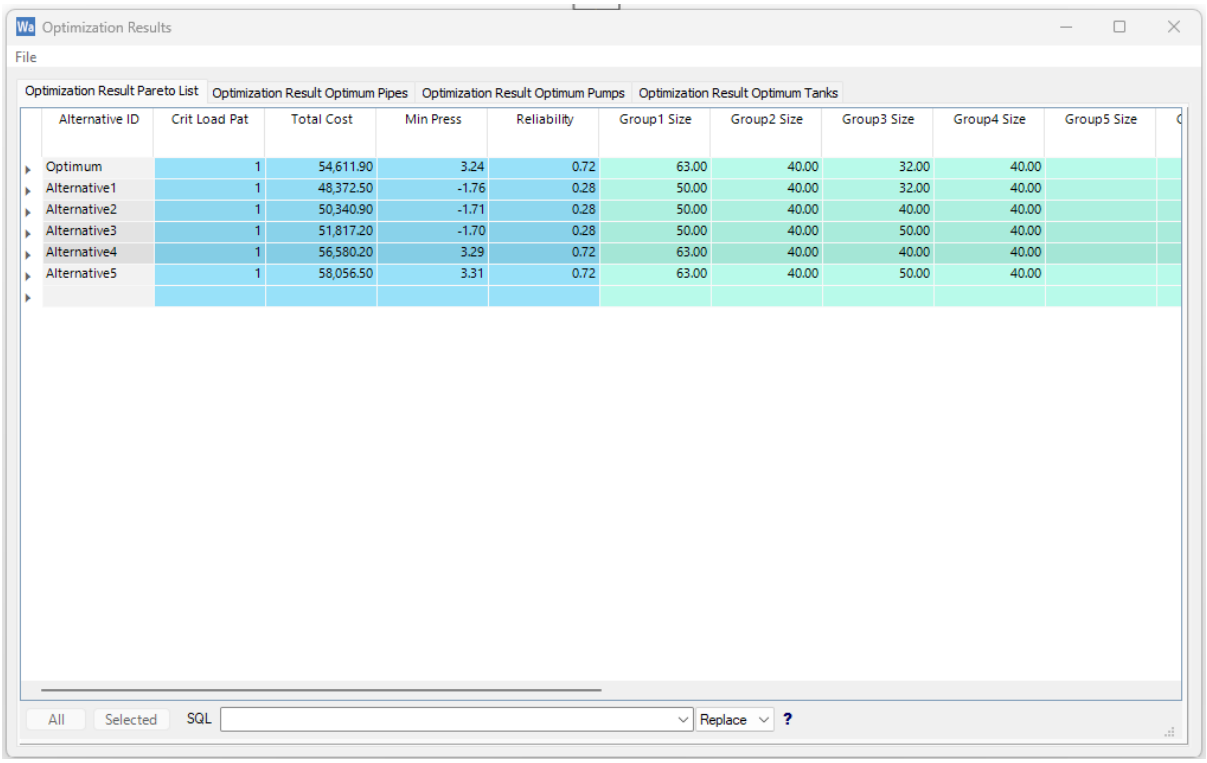
Layout Optimization

Index	Node Type	Node Cod	Exists	Elevation (m)	Output (L/s)	Emitter Coefficient	Sensitivity Output (L/s)	Sensitivity Group	AADD (kL/d)	AADD + UAW (kL/d)	Th AADD (kL/d)	Th AADD + Vac (kL/d)	Th AADD Zone (kL/d)
15	NODE	1	False	1,388.0	0.00	0.00			0.00	0.00	0.00	0.00	0.00
1	NODE	2	False	1,385.0	0.00	0.00			0.00	0.00	0.00	0.00	0.00
2	NODE	3	False	1,379.3	0.00	0.00			0.00	0.00	0.00	0.00	0.00
3	NODE	4	False	1,378.7	0.00	0.00			0.00	0.00	0.00	0.00	0.00
4	NODE	5	False	1,375.9	0.04	0.00			0.04	0.08	0.17	0.00	0.00
5	NODE	6	False	1,372.6	0.00	0.00			0.00	0.00	0.00	0.00	0.00
6	NODE	7	False	1,369.9	0.00	0.00			0.00	0.00	0.00	0.00	0.00
7	NODE	8	False	1,369.5	0.00	0.00			0.00	0.00	0.00	0.00	0.00
8	NODE	9	False	1,367.5	0.58	0.00			0.58	1.17	2.33	0.00	0.00
9	NODE	10	False	1,385.0	0.12	0.00			0.12	0.25	0.50	0.00	0.00
10	NODE	11	False	1,379.3	0.62	0.00			0.62	1.25	2.50	0.00	0.00
16	NODE	12	False	1,378.7	0.08	0.00			0.08	0.17	0.33	0.00	0.00
11	NODE	13	False	1,372.1	0.17	0.00			0.17	0.33	0.66	0.00	0.00
12	NODE	14	False	1,372.1	0.62	0.00			0.62	1.25	2.50	0.00	0.00
13	NODE	15	False	1,371.9	0.12	0.00			0.12	0.25	0.50	0.00	0.00
14	NODE	16	False	1,371.9	0.08	0.00			0.08	0.17	0.33	0.00	0.00
17	NODE	17	False	1,370.8	0.21	0.00			0.21	0.42	0.83	0.00	0.00
18	NODE	18	False	1,372.2	0.00	0.00			0.00	0.00	0.00	0.00	0.00
19	NODE	19	False	1,372.7	0.00	0.00			0.00	0.00	0.00	0.00	0.00
20	NODE	20	False	1,371.6	0.42	0.00			0.42	0.83	1.66	0.00	0.00
21	NODE	21	False	1,374.1	0.00	0.00			0.00	0.00	0.00	0.00	0.00
22	NODE	22	False	1,372.8	0.33	0.00			0.33	0.67	1.33	0.00	0.00
23	NODE	23	False	1,384.4	0.29	0.00			0.29	0.58	1.16	0.00	0.00

vi. Optimization of the network

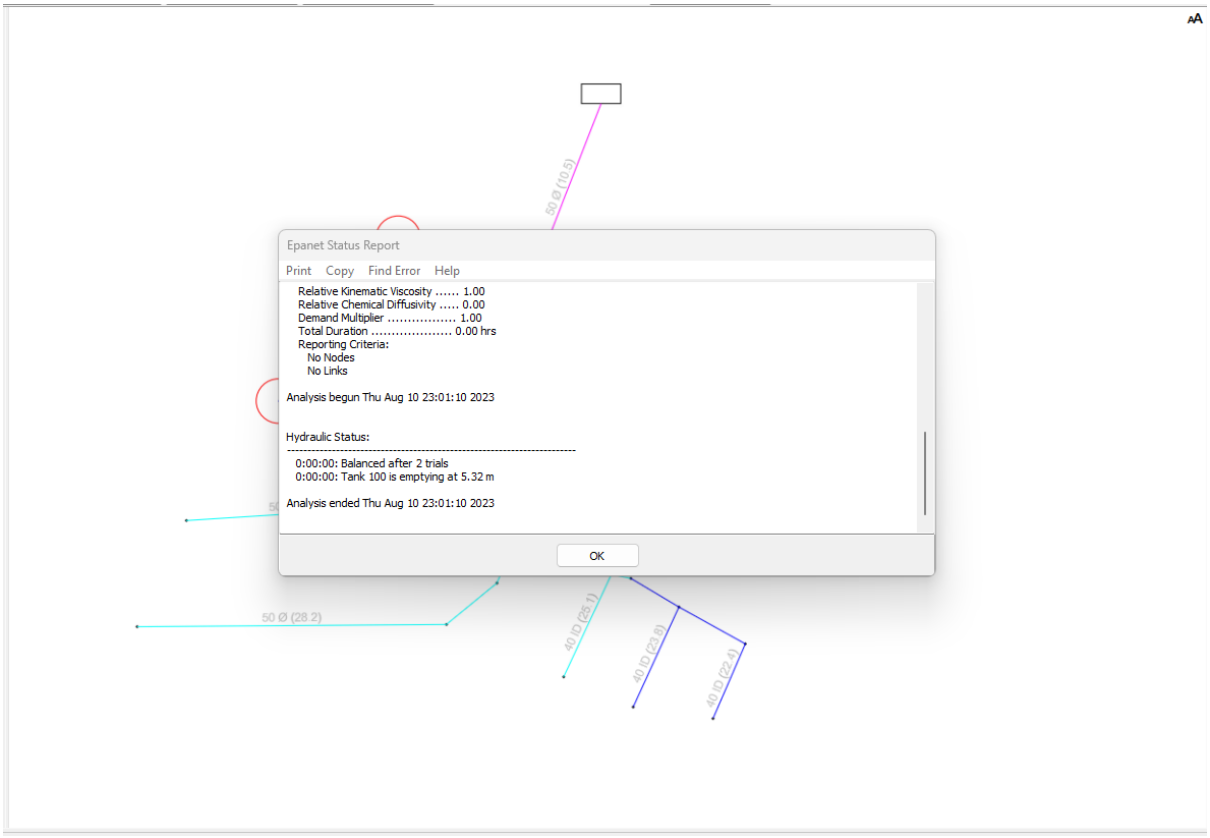


vii. Optimization results

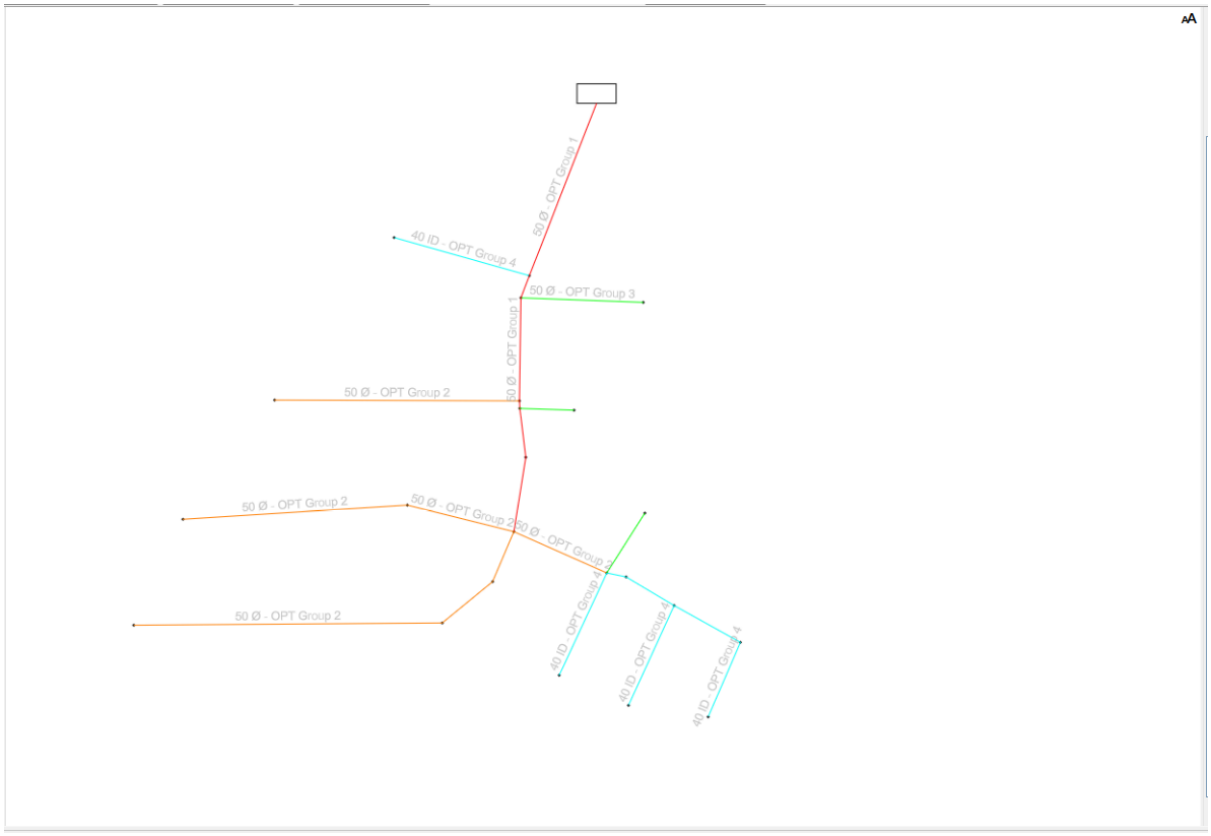


Alternative ID	Crit Load Pat	Total Cost	Min Press	Reliability	Group1 Size	Group2 Size	Group3 Size	Group4 Size	Group5 Size
Optimum	1	54,611.90	3.24	0.72	63.00	40.00	32.00	40.00	
Alternative1	1	48,372.50	-1.76	0.28	50.00	40.00	32.00	40.00	
Alternative2	1	50,340.90	-1.71	0.28	50.00	40.00	40.00	40.00	
Alternative3	1	51,817.20	-1.70	0.28	50.00	40.00	50.00	40.00	
Alternative4	1	56,580.20	3.29	0.72	63.00	40.00	40.00	40.00	
Alternative5	1	58,056.50	3.31	0.72	63.00	40.00	50.00	40.00	

viii. Applying ‘alternative 1’ optimization results



ix. Grouping of pipes and ‘alternative 1’ network



x. Flow and velocity results of ‘alternative 1’ network

__TswelopeleModelFinalSubmission.Wa_pipe

File Edit Utilities Table Configure Execute SQL

Layer __TswelopeleModelFinalSubmission.Wa_pipe - Joined " Chained "

Layout Results

Index	Link Type	Link Code	From Code	To Code	Exists	Diameter (mm)	Balanced Status	Flow (L/s)	Velocity (m/s)	Headloss (m)	Energy Gradient (m/m)	Average Head (m)
14	PIPE	1	100	1	False	50.0	OPEN	3.70	1.89	6.69	0.0886	10.5
15	PIPE	2	1	2	False	50.0	OPEN	3.41	1.74	0.72	0.0761	16.8
1	PIPE	3	2	3	False	50.0	OPEN	3.29	1.67	2.91	0.0710	19.3
2	PIPE	4	3	4	False	50.0	OPEN	2.66	1.36	0.14	0.0481	20.9
3	PIPE	5	4	5	False	50.0	OPEN	2.58	1.31	0.89	0.0454	22.1
4	PIPE	6	5	6	False	50.0	OPEN	2.54	1.29	1.31	0.0440	24.1
5	PIPE	7	6	7	False	40.0	OPEN	0.58	0.46	0.18	0.0086	26.3
6	PIPE	8	7	8	False	40.0	OPEN	0.58	0.46	0.22	0.0086	27.7
7	PIPE	9	8	9	False	40.0	OPEN	0.58	0.46	1.05	0.0086	28.2
8	PIPE	10	2	10	False	32.0	OPEN	0.12	0.16	0.07	0.0015	17.9
9	PIPE	11	3	11	False	40.0	OPEN	0.62	0.50	0.94	0.0097	20.2
16	PIPE	12	4	12	False	32.0	OPEN	0.08	0.10	0.01	0.0007	21.2
10	PIPE	13	6	13	False	40.0	OPEN	0.79	0.63	0.56	0.0150	25.0
11	PIPE	14	13	14	False	40.0	OPEN	0.62	0.50	0.86	0.0097	24.6
12	PIPE	15	6	15	False	40.0	OPEN	1.17	0.93	1.25	0.0308	24.8
13	PIPE	16	15	16	False	32.0	OPEN	0.08	0.10	0.02	0.0007	24.5
17	PIPE	17	15	17	False	40.0	OPEN	0.21	0.17	0.06	0.0013	25.1
18	PIPE	18	15	18	False	40.0	OPEN	0.75	0.60	0.12	0.0136	24.3
19	PIPE	19	18	19	False	40.0	OPEN	0.75	0.60	0.28	0.0136	23.7
20	PIPE	20	19	20	False	40.0	OPEN	0.42	0.33	0.20	0.0046	23.8
21	PIPE	21	19	21	False	40.0	OPEN	0.33	0.26	0.09	0.0030	22.6
22	PIPE	22	21	22	False	40.0	OPEN	0.33	0.26	0.09	0.0030	22.4
23	PIPE	23	1	23	False	40.0	OPEN	0.29	0.23	0.15	0.0024	17.3

xi. Nodal results of ‘alternative 1’ network

__TswelopeleModelFinalSubmission.Wa_node

File Edit Utilities Table Configure Execute SQL

Layer __TswelopeleModelFinalSubmission.Wa_node - Joined " Chained "

Layout Optimization

Index	Node Type	Node Cod	Exists	Elevation (m)	Output (L/s)	Simulated Output (L/s)	EGL (m)	Head (m)	Pressure (kPa)
15	NODE	1	False	1,388.0	0.00	0.00	1,403.6	15.6	153.2
1	NODE	2	False	1,385.0	0.00	0.00	1,402.9	17.9	175.6
2	NODE	3	False	1,379.3	0.00	0.00	1,400.0	20.7	203.0
3	NODE	4	False	1,378.7	0.00	0.00	1,399.9	21.2	207.5
4	NODE	5	False	1,375.9	0.04	0.04	1,399.0	23.1	226.2
5	NODE	6	False	1,372.6	0.00	0.00	1,397.7	25.1	245.8
6	NODE	7	False	1,369.9	0.00	0.00	1,397.5	27.6	270.4
7	NODE	8	False	1,369.5	0.00	0.00	1,397.3	27.8	272.4
8	NODE	9	False	1,367.5	0.58	0.58	1,396.2	28.7	281.5
9	NODE	10	False	1,385.0	0.12	0.12	1,402.8	17.8	174.9
10	NODE	11	False	1,379.3	0.62	0.62	1,399.1	19.8	193.8
16	NODE	12	False	1,378.7	0.08	0.08	1,399.8	21.1	207.3
11	NODE	13	False	1,372.1	0.17	0.17	1,397.1	25.0	245.2
12	NODE	14	False	1,372.1	0.62	0.62	1,396.2	24.1	236.7
13	NODE	15	False	1,371.9	0.12	0.12	1,396.4	24.5	240.4
14	NODE	16	False	1,371.9	0.08	0.08	1,396.4	24.5	240.2
17	NODE	17	False	1,370.8	0.21	0.21	1,396.4	25.6	251.0
18	NODE	18	False	1,372.2	0.00	0.00	1,396.3	24.1	236.3
19	NODE	19	False	1,372.7	0.00	0.00	1,396.0	23.3	228.7
20	NODE	20	False	1,371.6	0.42	0.42	1,395.8	24.3	237.9
21	NODE	21	False	1,374.1	0.00	0.00	1,395.9	21.8	214.0
22	NODE	22	False	1,372.8	0.33	0.33	1,395.8	23.0	225.7
23	NODE	23	False	1,384.4	0.29	0.29	1,403.5	19.0	186.8

APPENDIX 2: POPULATION GROWTH AND DESIGN FLOW RATES CALCULATION

YEAR	POPULATION	FACTOR (2.5 %)	DEMAND PER CAPITA	CONSUMPTION		25% RETICULATION LOSSES		DEMAND INCL LOSSES		PEAK FACTOR OF 1.5		INCLUDING RETICULATION FACTOR		TOTAL DEMAND	DEMAND PER HOUSE
			(l/d/person)	litres/day	m³/day	litres/day	m³/day	litres/day	m³/day	litres/day	m³/day	litres/day	m³/day	litres/second	litres/second
2015	534	0.025	80	42720	42.72	10680	10.68	53400	53.4	80100	80.1	192240	192.24	2.23	0.0250
	548	0.025	80	43840	43.84	10960	10.96	54800	54.8	82200	82.2	197280	197.28	2.28	0.0257
	562	0.025	80	44960	44.96	11240	11.24	56200	56.2	84300	84.3	202320	202.32	2.34	0.0263
	577	0.025	80	46160	46.16	11540	11.54	57700	57.7	86550	86.55	207720	207.72	2.40	0.0270
	592	0.025	80	47360	47.36	11840	11.84	59200	59.2	88800	88.8	213120	213.12	2.47	0.0277
2020	607	0.025	80	48560	48.56	12140	12.14	60700	60.7	91050	91.05	218520	218.52	2.53	0.0284
	623	0.025	80	49840	49.84	12460	12.46	62300	62.3	93450	93.45	224280	224.28	2.60	0.0292
	639	0.025	80	51120	51.12	12780	12.78	63900	63.9	95850	95.85	230040	230.04	2.66	0.0299
	655	0.025	80	52400	52.4	13100	13.1	65500	65.5	98250	98.25	235800	235.8	2.73	0.0307
	672	0.025	80	53760	53.76	13440	13.44	67200	67.2	100800	100.8	241920	241.92	2.80	0.0315
2025	689	0.025	80	55120	55.12	13780	13.78	68900	68.9	103350	103.35	248040	248.04	2.87	0.0323
	707	0.025	80	56560	56.56	14140	14.14	70700	70.7	106050	106.05	254520	254.52	2.95	0.0331
	725	0.025	80	58000	58	14500	14.5	72500	72.5	108750	108.75	261000	261	3.02	0.0339
	744	0.025	80	59520	59.52	14880	14.88	74400	74.4	111600	111.6	267840	267.84	3.10	0.0348
	763	0.025	80	61040	61.04	15260	15.26	76300	76.3	114450	114.45	274680	274.68	3.18	0.0357
2030	783	0.025	80	62640	62.64	15660	15.66	78300	78.3	117450	117.45	281880	281.88	3.26	0.0367
	803	0.025	80	64240	64.24	16060	16.06	80300	80.3	120450	120.45	289080	289.08	3.35	0.0376
	824	0.025	80	65920	65.92	16480	16.48	82400	82.4	123600	123.6	296640	296.64	3.43	0.0386
	845	0.025	80	67600	67.6	16900	16.9	84500	84.5	126750	126.75	304200	304.2	3.52	0.0396
	867	0.025	80	69360	69.36	17340	17.34	86700	86.7	130050	130.05	312120	312.12	3.61	0.0406
2035	889	0.025	80	71120	71.12	17780	17.78	88900	88.9	133350	133.35	320040	320.04	3.70	0.0416
	912	0.025	80	72960	72.96	18240	18.24	91200	91.2	136800	136.8	328320	328.32	3.80	0.0427
	935	0.025	80	74800	74.8	18700	18.7	93500	93.5	140250	140.25	336600	336.6	3.90	0.0438
	959	0.025	80	76720	76.72	19180	19.18	95900	95.9	143850	143.85	345240	345.24	4.00	0.0449
	983	0.025	80	78640	78.64	19660	19.66	98300	98.3	147450	147.45	353880	353.88	4.10	0.0460

No. of households is 89. Assume 6 people per household. Population as at 2015 is 89*6 = 534