CHARACTERISATION OF PRESSURE AND PHYSICAL WATER LOSSES IN WATER DISTRIBUTION SYSTEMS USING EPANET – THE CASE OF SOUTHERN REGION WATER BOARD, THYOLO BOMA

Master of Science in Sustainable Engineering Management (MSEM-Water) Thesis

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UNIVERSITY OF MALAWI

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By

Bright H. Piyo
(BSc Civil Engineering)

A thesis submitted in partial fulfilment of the requirements for a Master of Science degree in Sustainable Engineering Management - Water (MSs SEM-Water)

University of Malawi
The Polytechnic

December, 2016

DECLARATION

I declare that this research titled Characterisation of Pressure and Physical Water Losses in Water Distribution Systems Using EPANET, The Case of Southern Region Water Board, Thyolo Boma, is my own work. It is submitted in partial fulfilment of the requirements for the Master of Science Degree in Sustainable Engineering Management (Water) at the Polytechnic, University of Malawi. It has not been submitted for any other degree to any University.

SIGNATURE :

DATE :

CERTIFICATE OF APPROVAL

The undersigned certify that they have read and approve for acceptance by the University of Malawi, The Polytechnic this thesis titled, *'Characterisation of Pressure and Physical Water Losses in Water Distribution Systems using EPANET, The Case of Southern Region Water Board, Thyolo Boma'*.

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DEDICATION

I dedicate this thesis to my last born daughter Ekari, who despite her tender age, sat by dad's side pretending to be working on some serious write-up and at times refused to go to bed until I left too. She motivated me to push on.

ACKNOWLEDGEMENT

I thank the Almighty God for the good health and the gift of life to see me through this work when I felt like giving up. I also thank my academic supervisor, Mr Petros Zuzani for his untiring support, encouragement and guidance from conception through research work to compilation of the thesis. Thank you very much sir.

I also would like to acknowledge assistance from my friend, Eng. George Makungwa who played a great role in troubleshooting EPANET errors whenever I got stuck while conducting analyses. Great mention also goes to the following engineers for sharing both official and professional information on behalf of their organisations and themselves when asked; Steve Kazembe from Lilongwe Water Board, Andrew B. Masiye from Blantyre Water Board and Eng. Jacqueline Dias from Southern Region Water Board.

I am so grateful to Southern Region Water Board for sponsoring my studies financially and materially including allowing me to carry out research on one of their water supply systems which the centre of this thesis.

ABSTRACT

The study evaluates and characterises pressure in the existing Water Distribution System at Thyolo Boma. It investigates major factors causing pipe failures leading to high physical water losses with overall Non-Revenue Water of as much as 44%. The main objective was to reduce physical water losses through application of pressure management using EPANET models. Much focus was on the analysis of hydraulic regimes particularly dynamic operating pressure, unit head loss and velocity. A loss factor of 1.44, representing the system's UFW was applied on billed water consumption to determine base demand at nodes as EPANET input data. GPS was used to determine ground elevations. An analysis of faults register was carried out to determine rates of pipe bursts per kilometre per year (bursts/km/year) that is compared with pipe characteristics and EPANET results to establish correlations from which conclusions were drawn.

Results show that 42 percent of the distribution pipeline is in very high pressure zone while the rest is in low to moderately high pressure zones. Pipe bursts occur in high pressure zones with the exception of a few areas where pipe bursts emanate from poor pipe installation practices. High pressures are as a result of wide variations in topography. Application of some pressure management strategies like replacement of deteriorated pipes on a section of one of the high pressure zones proved effective in eliminating pipe bursts that consequently reduced leakage and hence reduction of physical water losses. In summary, leakage is primarily driven by differences in pressure in the distribution system. Introduction of pressure-break and pressure regulating facilities brought pressures to within permissible levels which will reduce pipe bursts, leakage and consequently physical water losses. Any effort to reduce excess pressure will reduce pipe failures that ultimately reduce physical water losses and thus improve operational efficiency.

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ABBREVIATIONS AND ACRONYMS

AC	Asbestos Cement
APA	American Psychological Association
ASCE	American Society of Civil Engineers
AWWA	American Water Works Association
BPT	Break Pressure Tank
BWB	Blantyre Water Board
DN	Nominal Diameter
EPA	US Environmental Protection Agency
FCMNRC	Federation of Canadian Municipalities and National Research Council
GI	Galvanised Iron
HDPE	High-density Polyethylene
ILI	Infrastructure leakage Index
IWA	International Water Association
IWRM	Integrated Water Resources Management
LWB	Lilongwe Water Board
NRW	Non-Revenue Water
NSO	National Statistical Office
OWL	Online Writing Lab
PAS	Performance Assessment Systems
PI	Performance Indicator
PN	Nominal Pressure
PRV	Pressure Reducing Valve
PVC	Polyvinyl Chloride
SRWB	Southern Region Water Board
UFW	Unaccounted for Water
UN	United Nations
USAID	United States Agency for International Development
WASAMA	Water Services Association of Malawi
WB	World Bank
WDM	Water Demand Management
WDN	Water Distribution Network

WDS	Water Distribution System
WTP	Water Treatment Plant

CHAPTER 1: INTRODUCTION

1.1 Background

Thyolo Boma Water Supply System, which falls under the jurisdiction of the Southern Region Water Board (SRWB) has been facing a number of challenges ranging from poor raw water quality particularly as a result of environmental degradation in the water catchment area and along the source river line (SRWB, 2006), inadequate production capacity (SRWB, 2008), aged or corroded pipes and steel tanks, and unregulated pressure (SRWB, 2007). SRWB cites these factors in their various reports as being the main cause for high rates of pipe failures leading to high Non-Revenue Water (NRW) of as much as 44 percent (SRWB, 2014) and unreliability of the Thyolo Water Distribution System (WDS) resulting in high rates and prolonged water supply interruptions, with some areas being affected for as long as 48 hours continuous (SRWB Mulanje, 2014a).

A review of the faults register at the utility's local office at Thyolo Boma in August 2014 showed that the rate of pipe bursts on some pipelines within the WDS is as high as 14.33bursts/km/year against the utility's standard of 0.55bursts/km/year (SRWB, 2015) rendering the distribution system as being of low operational efficiency and consequently of high operational costs, therefore impinging on the utility's profitability and customer service in general. Even compared to other utilities within the country, the rate is still very high. For instance, Blantyre Water Board (BWB) has a permissible burst rate of 0.9bursts/km/year (BWB, 2015) while Lilongwe Water Board (LWB) has 0.3bursts/km/year (LWB, 2014). SRWB cites aged pipeline, corrosion, unregulated high operational and transient pressures in the distribution system, poor pipeline design or construction (low nominal pressure pipes and low pipe trenches exposing pipes crushing from surcharge and vandalism) as being the main factors causing high pipe failure rates leading to high physical water losses (SRWB Mulanje, 2014b).

Previous studies on leakage by Lambert (2002), and Thornton and Lambert (2011) indicate that "leakage is positively related to pressure and hence reducing pressure immediately reduces leakage". Thornton (2003) also came up with a study that showed that "rate of leakage in a WDS is a function of pressure applied by either pumps or gravity head and that pipe burst is a function of pressure". Long term failure behaviour, the tendency of a water pipe to burst is considered to depend on fixed parameters such as pipe material, pipe diameter, pipe age, system pressure and transient pressure events (Skipworth, et al., 2002). For instance, pressure resistance of Asbestos Cement (AC) pipes deteriorates linearly to a factor of 10.5 for pipes aged above 40 years (Mordak & Wheeler, 1988). Similarly, the rate of deterioration varies widely as it depends on physical, environmental and operational conditions (Jafar, Shahrour, & Juran, 2010). A study by Farley and Trow (2003) established that water pressure management extends pipe's lifespan. However, Mutikanga, et al. (2013) observed that water distribution networks (WDN) for most water utilities, particularly those with limited financial resources, are usually not well configured for effective pressure management. This is true for Thyolo Boma WDN.

It is on the basis of the above outlined challenges that an idea was conceived to evaluate the hydraulic regimes in the existing Thyolo Boma WDS using EPANET 2.0, for the purpose of characterising the regimes to correlate them with pipe bursts patterns and their causes leading to high physical water losses. The main areas focus for characterisation of the hydraulic regimes included pipe type, size, age, and working pressure. The findings would give clues on how to optimise the distribution system that would consequently be solutions for reducing or eliminating pressure induced pipe bursts and hence physical water losses.

Rossman (2000) describes EPANET as a computer program that performs extended period simulation of hydraulic and water quality behaviour within pressurized pipe networks. It tracks the flow of water in each pipe, the pressure at each node, the height of water in each tank, and the concentration of a chemical species throughout the network during a simulation period comprising multiple time steps. In addition to chemical species, water age and source tracing can also be simulated. It is basically designed to be a research tool for improving the understanding of movement and fate of drinking water constituents within water distribution systems. It can also be used for many different kinds of applications in distribution systems analysis such as sampling program design, hydraulic model calibration, chlorine residual analysis, and consumer exposure assessment. Running under windows, EPANET provides an integrated environment for editing network input data, running hydraulic and water quality simulations, and viewing the results in a variety of formats. These include colour-coded network maps, data tables, time series graphs, and contour plots. EPANET has successfully been applied in similar circumstance before. For instance, Adeniran and Oyelowo (2013) used it to carry out a comprehensive analysis of 30-year-old existing water distribution system for the University of Lagos in Nigeria whose population and water demand had risen from 12,000 to 85,000 people and 2.48 million litres per day (mlpd) to 10.75mlpd respectively between 1982 and 2012 rendering the system's performance inefficient. From the model analysis, a recommendation was made to raise the level of the main service reservoir and WDS managed to achieve optimal hydraulic performance (adequate pressure heads, flow velocities and low unit head losses) with just a few pipes registering low water flow rates.

Vyas, et al. (2013) used EPANET to assess the performance of Dhrafad Regional Water Supply Scheme in order to optimise it to address any improvements required to the existing infrastructure and the mode of operation to improve quality and quantity of water distributed to consumers. The optimised model gave output results with minimum head losses and economical diameters. It followed that appropriate pressure heads were achieved including desired flows at sufficient velocities through the distribution pipe network. Shen (2007) implemented a new water quality model in EPANET with the aim of reducing Arizona Public Drinking Water Systems' vulnerability to contamination. He specifically modified EPANET's water quality model to drastically improve the Arizona city's effectiveness in monitoring the water supply system and ensure that residents had access to the cleanest water in the world. Ingeduld, et al. (2006) used EPANET to model intermittent water supply systems. EPANET source code was adjusted to allow for modelling pressure dependent demands for dealing with low pressure and "dry pipe" situations. The solution was found to be robust, simple and proved to be useful and practical for modelling "dry pipes".

The above scenarios, where EPANET has successfully been used to carry out comprehensive hydraulic and water quality analyses, are similar to that of Thyolo Boma WDS. It therefore follows that EPANET can effectively be applied to assess the current hydraulic performance of the existing Thyolo Boma WDS to establish the linkage between the hydraulic regimes and pipe burst rates. The same EPANET shall be used to model an optimised WDS in which most or all hydraulic regimes such as pressures, flow velocities and unit head losses are brought to within permissible levels. Based on Lambert (2002) and Thornton and Lambert (2011), reduction in high pressures shall consequently lead to reduced leakage hence reduced physical water losses in the distribution system.

1.2 Study Area

The study area is Thyolo district administrative headquarters, which is located to the south east of Blantyre district. It is 34km from Limbe central business district to the district headquarters, Thyolo Boma. The choice of Thyolo Boma as a town as a case study for this research was basically due to four factors:

Firstly, it was convenient given that the author has direct contact and access to utility personnel, data and water distribution infrastructure. Secondly, it has high levels of Non-Revenue Water that required urgent attention for any available solutions to reduce the water losses at as fairy low cost as possible. Thirdly, the distribution system is small and therefore manageable in terms of complexity and timely collection of data to meet research timelines. Lastly the poor state of the water distribution system infrastructure in most parts that appears to have outlived its design span and therefore requiring attention. The study area has been presented in two ways; at national level and district level as shown in Figure1 and Figure2.

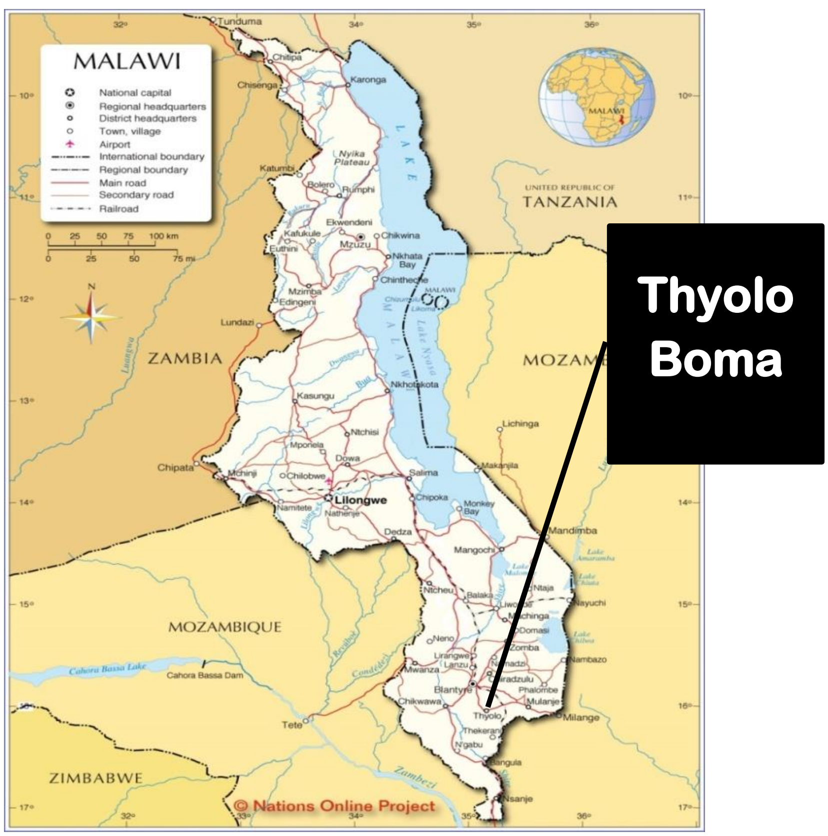

Figure 1: Map of Malawi Showing the Location of Thyolo District

Source: *Nations Online Project*

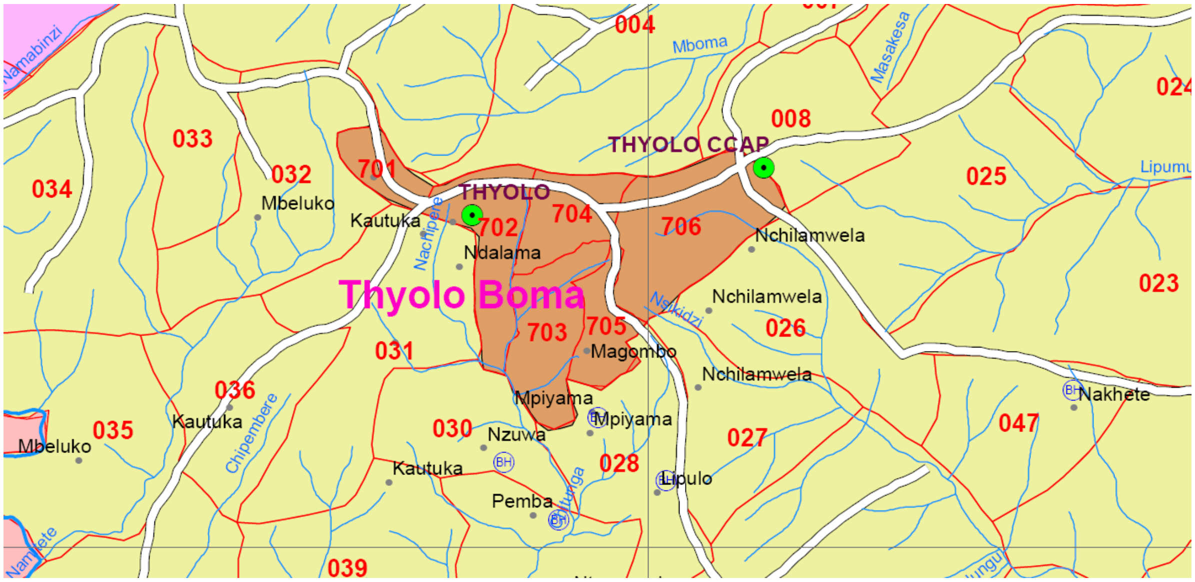

Figure 2: Part map of Thyolo District Showing Thyolo Boma (Coloured Brown), the Study Area

Source: National Statistical Office (NSO), Zomba, Malawi

1.3 Motivation for the Study

As the world continues to urbanise, sustainable development challenges will be increasingly concentrated in cities, particularly in the lower-middle-income countries where the pace of urbanization is fastest (United Nations, 2014). Due to Africa's and Malawi's rapid urbanisation in particular (Figures 3 and 4) and water scarcity, maintaining a stable and safe water supply has become a challenge to all cities and towns in Malawi, whereas a large amount of water is lost or wasted from the pipes of the distribution systems while at the same time about a quarter of residents are yet to be served with the potable water. By 2011, Blantyre Water Board (BWB) was only able to supply about 79,410 m^3 of water per day to Blantyre city (UN-Habitat, 2011a) against a demand of 96,000 m^3 per day (Magombo & Kosamu, 2016). The water utility had non-revenue water (NRW) as much as 49% and service coverage of 75% by 2016 (Magombo & Kosamu, 2016). Their supply network is old with frequent pipe bursts that require constant maintenance which is expensive while on the other hand demand for water had surged with the growing population resulting in acute water shortages in the city (UN-Habitat, 2011a). Lilongwe Water Board (LWB) only supplied between 69,000 and 78,000m3 of water per day, with the daily demand estimated at 73,250m3 of water at service coverage of 75%in 2008 while water losses were at about 44 percent (UN-Habitat, 2011b). Water supply in Mzuzu is unevenly distributed. The informal settlements are the most affected having little or no access to water services. They rely on communal water points (kiosks) where water supply is unreliable. About 13.4 percent of informal settlements' residents acquire their water from unprotected water sources, such as wells, rivers and streams (UN-Habitat, 2011c). In Zomba city, the Southern Region Water Board registered water losses of 38.1% in 2014 (SRWB, 2014), and 37.2% in 2015 (SRWB, 2015b) for service coverage of 81% and 82% respectively signifying some slight improvement. The water leakage is not only a waste of water resources, but also results in great socio-economic costs (Xu, Liu, Chen, & Li, 2014). This is expected to exert more pressure on the limited water resources and financial resources for capital investments in water infrastructure expansion to meet corresponding rise in urban water demand. Malawi is considered a water-stressed country, and per capital water availability is rapidly declining due to remarkable population growth, especially in its urban and peri-urban areas. Besides the water stress, challenges to urban water supply in Malawi include aging water systems, high levels of non-revenue water and low cost recovery within utilities (USAID, 2010). Therefore reduction in physical water losses through various available strategies, pressure management for instance, in high pressure water distribution systems like that of Thyolo, can ease the water stress and allow the water resource to reach more consumers while at the same time increasing the utility's profitability and sustainability of its Water Distribution Systems (WDS) in light of Malawi's rapid urbanisation and population growth that is triggering corresponding rise in water demand.

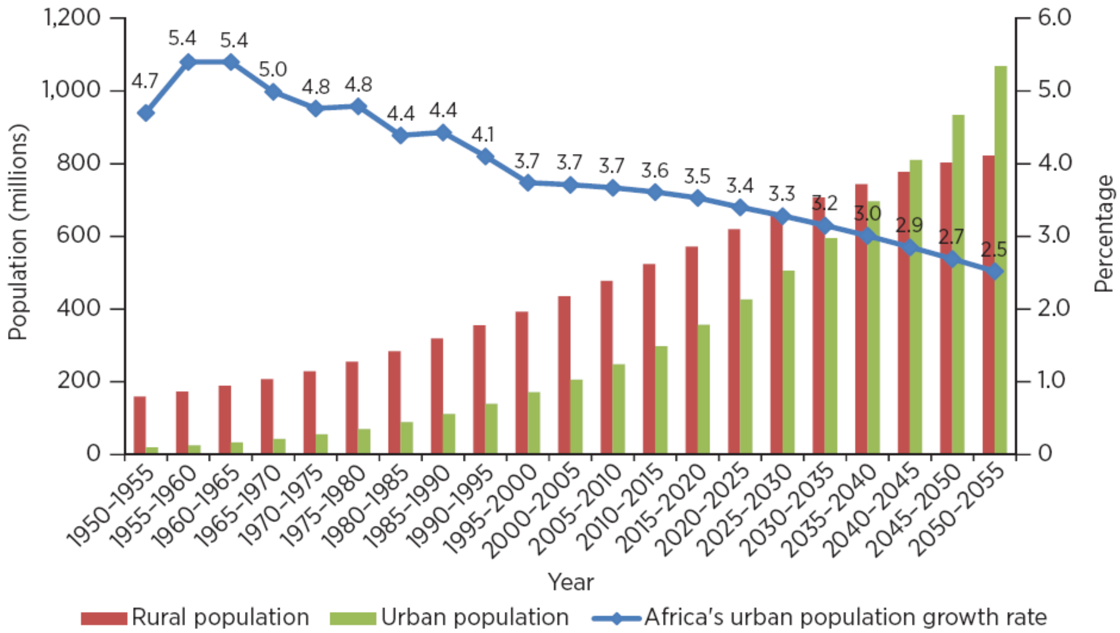

Figure 3: Trends in Urbanisation in Africa

Source: UNDESA, 2012

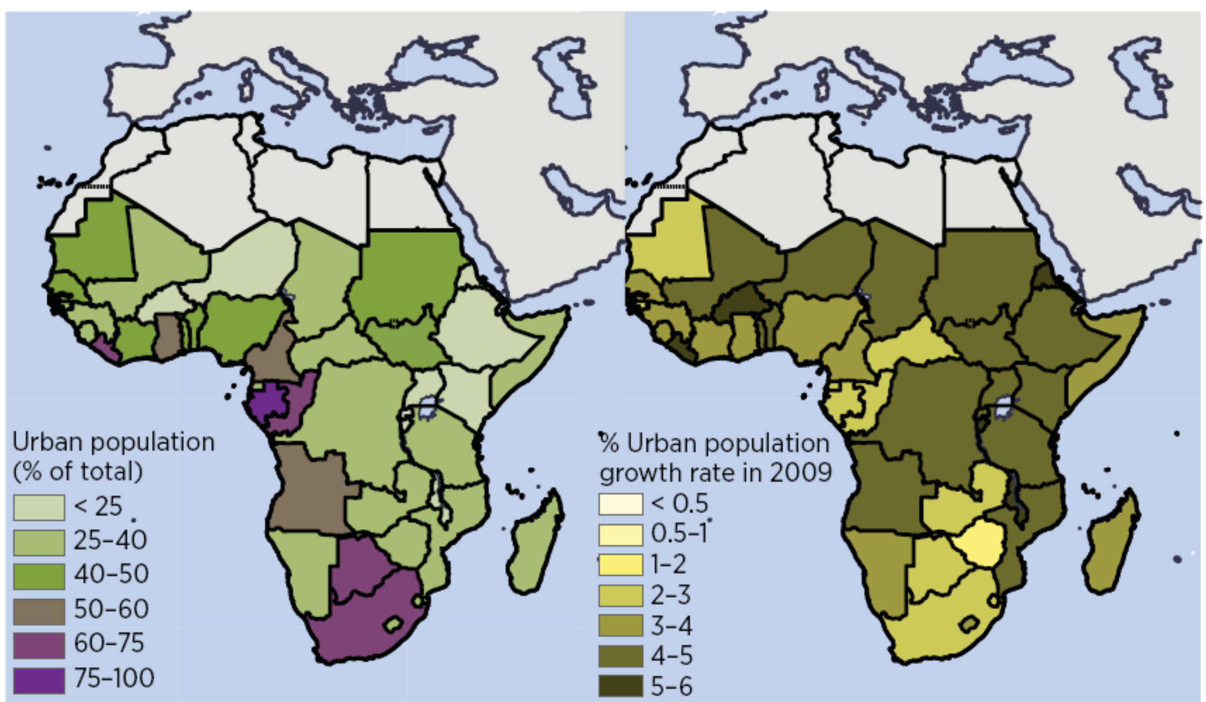

Figure 4: Level of Urbanisation and Urban Population Growth Rates in Africa

Source: Africa Spatial Services Helpdesk based on World Bank, 2010a and 2011a

Pressure management is one of the most important water demand management interventions that can be implemented by a water utility in its efforts to reduce leakage. Since leakage is driven by pressure, any efforts which result in the reduction of water pressure for even part of the day will reduce leakage to some extent (McKenzie and Wegelin, 2009). In this regard, unregulated pressure in a WDS with wide varying topography like that of Thyolo Boma, can cause serious problems to transmission and distribution pipes' capacity to contain high hydrostatic and hydraulic heads in some sections of the distribution system.

1.4 Problem Statement

Southern Region Water Board, like all other five Water Boards in Malawi, faces a number of challenges particularly with regard to operational efficiency (UN-Habitat, 2011a), (UN-Habitat, 2011b), (UN-Habitat, 2011c), (UN-Habitat, 2011d). Thyolo water supply, located within and around Thyolo District administrative headquarters (hereinafter referred to as Thyolo Boma), falls under Southern Region Water Board, under one of its five administrative units known as Mulanje Zone.

The Zone comprises seven water supply centres namely Mulanje, Muloza, Luchenza, Phalombe, Thyolo, Mikolongwe and the Malawi University of Science and Technology (MUST). With the exception of MUST and Muloza that are relatively new systems at 2 years and 13 years respectively, the rest are very old ranging from 30 to 42 years of operation as at year 2015, manually operated and highly inefficient (SRWB Water Reticulation Maps), (SRWB, 2013). Monitoring of the distribution systems involves physically going along pipeline routes, tanks and related facilities to check the condition of the facilities including water quality parameters like turbidity, residual chlorine levels and faults such as pipe bursts, tank overflows or pipe leaks. This kind of monitoring is time consuming and ineffective besides being costly in terms of human resources, transportation and time.

Thyolo and Mulanje, of all the water distribution systems listed above, are the oldest with 42 years in operation as at year 2015. However, Thyolo has had little rehabilitation compared to the rest particularly in terms of replacement of aged and dilapidated pipes and upgrading of small pipelines that have become almost obsolete over the years as a result of declining carrying capacity and increasing rate of failure. Consequently, unaccounted for water (Non-Revenue

Water), currently averaging 44% (SRWB, 2014) is very high, a development that denies consumers consistent water supply through water interruptions arising from mains and reservoir failures. Maintenance costs (Figure 5) for the distribution system, based on pipe maintenance materials and labour, were relatively very high for such a small WDS compared to other much larger systems within the Zone like Mulanje Boma WDS. The pipeline maintenance costs, coupled with high water loss, create unnecessary high water demand thereby putting a strain on limited water resources (SRWB Mulanje, 2014b).

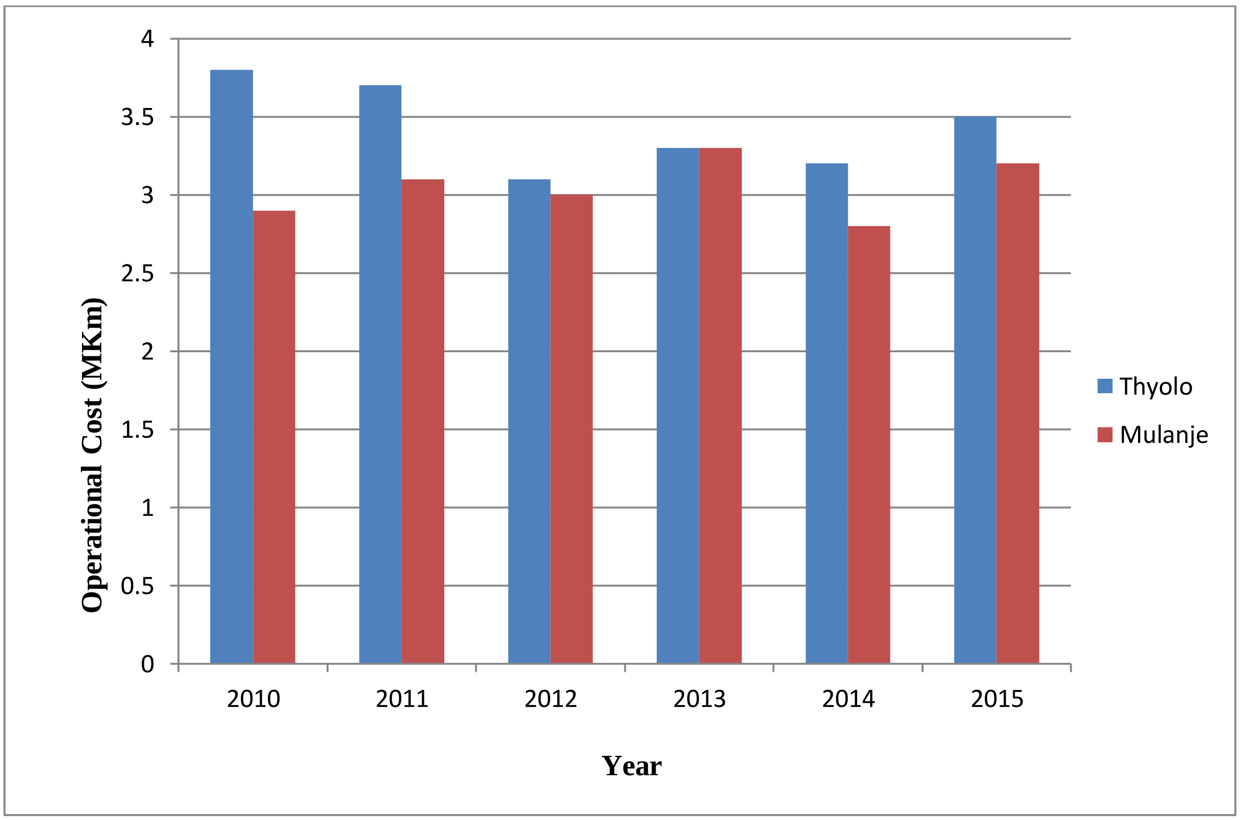

Figure 5: Pipeline Maintenance for Thyolo Boma and Mulanje Boma WDS

Thyolo Boma water distribution system experiences high number of pipe bursts and leakage leading to high water losses of as much as 44% indicated above. The Board's 2013/2014 annual report shows that Thyolo registered 699 pipe breakdowns between July 2013 and June 2014 translating to about 2 pipe breakdowns per day. Figure 5 above compares pipeline maintenance costs for Thyolo and Mulanje Boma water distribution systems as extracted from fault registers and annual operational reports for the centres. It can be observed that Thyolo, despite being almost half the size of Mulanje, had higher maintenance costs of the two. It can further be observed that the costs for Thyolo reduced between 2012 and 2014 when parts of Glennae and Kasembereka pipelines were replaced by new and high pressure rated pipes.

For a 55km long distribution network, the 699 one-year pipe breakdowns translate to 12.71 bursts per kilometre per year. The Board's performance indicator for bursts is 0.55bursts per kilometre per year (SRWB, 2015). Other similar water utilities such as Lilongwe Water Board's (LWB) standard are 3 leaks and 0.3bursts/km/year (LWB, 2014)while Blantyre Water Board (BWB) is 0.9bursts/km/year (BWB, 2015). It appears there is no national standard on this performance indicator yet and therefore each water utility has developed its own target. Average unaccounted for water by the end of this period in June 2014 was 32.1% with the highest recorded loss being 43.8% in June 2014 and the lowest 21.5% in September 2013 (SRWB, 2014).

Globally, studies indicate that up to 50 to 60 percent of treated and pumped water is lost in transit from water treatment plant to consumer end (Dighade, Kadu, & Pande, 2014). Locally, SRWB registered 32.3 percent Non-Revenue Water (NRW) (SRWB, 2014), LWB had 35 percent (UN-Habitat, 2011b) while BWB had up to 49 percent (Magombo & Kosamu, 2016). High levels of NRW reflect huge volumes of water lost through physical and apparent loss components not being invoiced to customers. This seriously affects the financial viability of water utilities through lost revenues and increased operational costs. The overall objective of any water distribution system is to deliver wholesome water to consumers at adequate residual pressure in sufficient quantity, quality and achieve continuity and maximum coverage by reducing water losses. High frequency of pipe bursts has compromised this objective. Consumers at times complain of soiled water when water supply is being restored after an interruption possibly due to inadequate scouring of dirty ingress water in the maintenance trench that compromises water quality; no running taps as broken mains are being maintained which forces those consumers without adequate storage to source water from either far distances or unsafe sources. Such incidents are common in Thyolo WDS that subsequently trigger high operational costs and high NRW levels. Therefore, Thyolo water distribution system in its current state is highly unreliable and inefficient.

1.5 Research Objectives

The study seeks to evaluate current hydraulic regimes of Thyolo Boma water distribution system to see if they are within permissible ranges for effective operational efficiency or outside permissible ranges that may be causing pipe failures and hence high non-revenue water. The following are the objectives.

1.5.1 General Objective

The main objective of this study was to characterise the water distribution system of Thyolo Boma so as to provide recommendations that could be implemented to optimise it for reduction of physical water losses.

1.5.2 Specific Objectives

In order to achieve measurable results from the general objective, the following specific objectives were developed:

i. Identify vulnerable areas prone to pipe breakages
ii. Determine hydraulic regimes (e.g. water pressure, flow velocity and unit head loss) in the distribution system using EPANET 2.0 model.
iii. Establish factors leading to frequent pipe bursts

1.6 Research Questions

In order to achieve the objectives, the study had to focus and respond to the following questions to properly guide the research.

i. How does the pressure distribution pattern relate to the frequency of pipe bursts?
ii. Does elevation (potential energy), flow velocity (kinetic energy), and pressure at a particular position in the distribution system contribute to pipe bursts?
iii. Is age of a pipe a contributing factor to frequency of pipe bursts?
iv. Does pipe size, type, nominal pressure (pipe pressure rating) and corrosion relate to frequency of pipe bursts?

The next chapters will therefore outline more details on previous studies carried out by others related to this study, the methodology employed to achieve the objectives of this study, results of the study and discussion thereto, conclusions and recommendations as well as recommendations on areas for further research arising from the study.

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

This chapter conducts and composes a thorough literature review that will define and refine the research topic, gather basic background information, search and locate related published books and articles (including related threads of research) that will be analysed and synthesized into review form. The purpose is to identify similarities, contrasts and gaps in previous research studies of similar nature to guide and refine this topic. The goal is to create new knowledge.

2.2 Water Distribution Systems

A Water Distribution system (WDS) is a network of pumps, pipes, storage tanks, valves, reservoirs, water meters, pipe fittings and other hydraulic appurtenances spanning from a few kilometers to thousands of kilometers depending on the size of the area or town being supplied. Its function is to carry and transport drinking water from a centralized Water Treatment Plant (WTP) or well supplies to consumers' taps (US Academy of Sciences, 2006). A well planned and functioning WDS must deliver the required water quality and quantity at a suitable pressure. Failure to do so is a serious deficiency.

2.2.1 Water Distribution Pipe Network Design and Optimisation

There are many factors that affect the design of a water distribution system. They include environmental, financial and legal challenges to name but a few. In cases where the limiting factor is financial, the number of resources used in a design would be limited in order to limit costs (Abbott, 2012). Countries that have limited financial resources may design their WDS using minimum amount of materials required to meet the demands of a system. In most such circumstances, countries or utilities mainly opt for branched pipe network in their WDS for purposes of reducing costs (Jordan Jr., 2006 and Mihelcic, et al., 2009). This entails that each demand is supplied by one pathway only. Branched pipe networks may suffice in cases of economic planning or urgent remedy, sufficient in meeting water demands as long as there are no pipe failures, leaks or damage to any part of the system (Cembrowicz, 1992).In case of failure in the system in a branched distribution network, nodes downstream of the damage would be without water supply. Providing redundant pathways, like in looped pipe networks, is ideal but may not be feasible for every demand node (Abbott, 2012).

Where finances are not a limiting factor, WDS are more robust. Robustness, as defined by Yazdani and Jeffrey (2012), is the optional connectivity of a network to reduce the probability of between the hydraulic failures or to reduce the consequences of component failures. This is achieved through the addition of loops to the system. Loops provide redundancy and therefore increase reliability of the WDS. Redundancy is provided by the addition of alternative paths between the source and demand nodes that that can be used to satisfy water supply requirements during failure of the main paths (Goulter, 1987). Increasing paths between demand and sources increase probability that a system will be operational, and that demand is met which are two of the definitions for reliability (Baranowski, et al., 2003).

Therefore, branched pipe network systems and partially looped networks with limited redundancy or no redundancy at all are more likely to have outages due to leak, breakages, or other component failures. Looped networks also absorb pressure surges better than branched networks as there is less flow restriction compared to branched networks where water hammers are generated from the dead ends at pipeline end caps.

2.2.2 Optimisation of Operation, Monitoring and Management of Water Distribution Systems

In order to improve the efficiency of WDS operation and management, water utility companies are increasingly embarking on long term monitoring programs or conducting data logging to capture the hydraulic and water quality characteristics throughout the water network. The data collected can be used for many purposes. For instance, pressure and flow data can be used for detecting anomaly events, predicting water consumption, calibrating hydraulic model, identifying new and long-lasting leakage hotspots (Wu, Song, & Roshani, 2015).

2.2.2.1 *Hydraulic Models for Pressure and Leak Reduction through Flow Regulation*

Araujo, et al. (2006) developed a model to support decision systems regarding the quantification, location and opening adjustment of control valves in a water pipe network system. The main objective was to minimise pressures and consequently develop leakage levels. The study aimed at establishing a solution that allows simultaneous optimisation of a number of valves and their locations, as well as valve opening adjustments for simulation in an extended period, dependent of system characteristics. EPANET was used for hydraulic network analysis, and two operational models were developed based on Genetic Algorithm optimisation method for pressure control and hence leakage reduction since leakage is a pressure dependent function. The two models guaranteed an adequate technique performance but Araujo, et al. (2006) proposed the need for global evaluation of the system for different scenarios, by means of operational conditions for different restrictions of each component. Araujo, et al. (2006) highlight previous similar studies presented by Jowitt and Xu (1990), Alonso, et al. (2000), Vitkovsky, et al. (2000), and Ulanicka, et al. (2001) on minimisation of pressure as a conditional parameter of leak indicator in water network systems. As regards methodology employed in reduction of pressures, Jowitt and Xu (1990), Reis and Porto (1997), Kalanithy and Lumbers (1998), Reis and Chaudhry (1999), Tucciarelli, et al. (1999), and Ulanicka, et al. (2001), suggested that the best solution must include the use of elements which provoke head losses, such as pressure reducing valves (PRVs). Vairavamoorthy and Lumbers (1998) analysed the optimum localisation of valves which Araujo, et al. (2006) regarded as a crucial step being reviewed in their research.

Araujo, et al. made four conclusions from the study. That EPANET model for hydraulic simulation and two operational models allow optimisation of the number and location of control valves as well as their opening adjustments, for an effective optimisation of leakage levels since leaks are modeled as orifice by pressure dependent functions. That pressure and leakage distribution along the water distribution system, as well as operational status of each installed valve allows to compare possible solutions and to estimate the average leakage gain. That selection of the best number and location of possible candidate valves depends on topography and characteristics of the system, which is only obtained by computational sensitivity analysis.

2.2.2.2 *Hydraulic Models for Water Loss Reduction through Pipe Bursts Localisation*

Pipe bursts are one of the leading causes of NRW. Reducing water loss through pipe bursts is therefore a major challenge throughout the developed and developing world. Current burst lifetimes are often long because awareness and locations of such is time and labour intensive (Farley, Mounce, & Boxall, 2012). Advances that can reduce these periods will lead to improved leakage performance, customer service and reduced resource wastage.

Farley, et al. (2012) carried out a study on development and field validation of burst localisation methodology. It was based on the theory that in a WDS, the sensitivity of a pressure instrument to change, including burst events is greatly influenced by its own location and that of the event within the network. In the study, a method is described that utilises hydraulic model simulations to determine the sensitivity of potential pressure instrument locations by sequentially applying 'leaks' to all potential burst locations. Results from the field studies that demonstrated practical application of the method showed that current standard network models can provide sufficient accurate quantification of different sensitivities and that once combined with event detection techniques for data analysis, events can effectively be localized using small number of instruments.

2.2.3 Deterioration of Water Distribution Mains and Burst Rate

Federation of Canadian Municipalities and National Research Council (FCMNRC) (2003) classified pipe failure cause factors into three namely physical, environmental and operational factors. It compiled a summary of the factors leading to deterioration of water distribution mains that cause water distribution pipes to burst under dynamic pressure as presented in Table 1.

2.2.3.1 *Deterioration of Water Mains*

Deterioration of a water main is described as a general decline of its physical state. While most of previously published work has focused upon pipe, material properties and environmental conditions, there is little understanding of the extent operational conditions such as sudden and gradual pressure fluctuations are a contributing factor for pipe deterioration and failures (Rezaei, Ryan, & Stoianov, 2015). The rate of deterioration varies widely as it depends on physical, environmental and operational conditions (Jafar, Shahrour, & Juran, 2010). Further to this, the combination of factors that lead to pipe deterioration varies among different pipe networks (Rajani and Tesfamariam, 2005; Wood and Lence, 2006a).

Although significant work has been placed into modelling pipe deterioration, a comprehensive model is yet to be developed due to the complex processes involved, variable environmental conditions and lack of relevant data (Kleiner, Adams, & Rogers, 2001). For this reason, the majority of researchers adopt statistical methods, data driven methods and evolutionary techniques for the analysis of pipe failure and deterioration (Kleiner, et al., 2001). Barbados Water Authority has specified the following factors that should be considered in the model; pipe material, pipe diameter, pipe installation era, maximum demand flow, user connection density, traffic loading, soil type, and slope. These factors were selected based on the availability of data and potential of these factors contributing to the model. The rest of factors that cause deterioration of water distribution mains are detailed in Table 1 below.

Table 1: Factors that Contribute to Water Distribution System Deterioration

FACTOR		EXPLANATION
Physical	Pipe material	Pipes made from different materials fail in different ways
	Pipe wall thickness	Corrosion will penetrate thinner walled pipe more quickly
	Pipe age	Effect of pipe degradation become more apparent over time
	Pipe vintage	Pipes made at a particular time and place may become more vulnerable to failure.
	Pipe diameter	Small diameter pipes are more susceptible to beam failure.
	Type of joints	Some types of joints have experienced premature failure (e.g. leadite joints)
	Thrust restraint	Inadequate restraint can increase longitudinal stresses
	Pipe lining and coating	Lined and coated pipes are less susceptible to corrosion
	Dissimilar metals	Dissimilar metals are susceptible to galvanic corrosion
	Pipe installation	Poor installation practices can damage pipes, making them vulnerable to failure.
	Pipe maintenance	Defects in pipe walls produced by manufacturing errors can make pipes vulnerable to failure. This problem is most common in order pit cast pipes
Environmental	Pipe bedding	Improper bedding may result in premature pipe failure
	Trench backfill	Some backfill materials are corrosive or frost susceptible.
	Soil type	Some soils are corrosive, some soils experience significant volume changes in response to moisture changes, resulting in changes to pipe loading. Presence of hydrocarbons and solvents in soil may result in some pipe deterioration.
	Groundwater	Some groundwater is aggressive toward certain pipe materials.
	Climate	Climate influences frost penetration and soil moisture.
	Pipe location	Migration of road salt into soil can increase the rate of corrosion
	Disturbances	Underground disturbances in the immediate vicinity of an existing pipe can lead to actual damage or changes in the support and loading structure on the pipe.
	Stray electrical	Stray currents cause electrolytic corrosion

	currents	
	Seismic activity	Seismic activity can increase stresses on pipe and cause pressure surges.
Operational	Internal water pressure, transient pressure	Changes to internal water pressure will change stresses acting on the pipe
	Leakage	Leakage erodes pipe bedding and increases soil moisture in the pipe zone
	Water quality	Some water is aggressive, promoting corrosion
	Flow velocity	Rate of internal corrosion is greater in unlined dead-ended mains
	Backflow potential	Cross connections with systems that do not contain potable water can contaminate water distribution systems.
	O&M practices	Poor practices can compromise structural integrity and water quality.

Source: *Federation of Canadian Municipalities and National Research Council (2003), Issue No. 1.1*

2.2.3.2 *Pipe Burst Rate*

Pipe burst rate is the frequency at which a water transmission or distribution pipe fails per given length and period. The rate is normally expressed as number of bursts per kilometre per year (bursts/km/year). Pipe bursts are caused by applied forces exceeding the residual strength of a pipe material. Pipe breakage occurs when the stresses (operational and environmental) act on pipes where corrosion, degradation, inadequate or poor installation or manufacturing defects has impacted the pipes' structural integrity (Rezaei, Ryan, & Stoianov, 2015). Skipworth, et al. (2002) pointed out that for long term failure behaviour, the tendency of a water pipe to burst can be considered to depend on fixed parameters such as pipe material, pipe age, pipe diameter, system pressure, transient pressure events, density of consumer service lines, soil parameters, traffic loading, water temperature, ground type and water quality.

A study by Rezaei, et al. (2015) established that among all pipe materials commonly used in WDSs, cast Iron (CI) has the highest burst rate followed by Asbestos cement (AC), Polyvinyl chloride (PVC), Polyethylene (PE), and lastly ductile iron (DI). Brittleness of cast iron and its lower tensile strength were cited as probable main reason for the higher density of failures. Rezaei, et al. (2015) further established that reduction in burst rate per length of a pipeline. In Malawi, the most commonly used pipe materials, particularly for diameters less than 350mm, are PVC, galvanised iron (GI), High density polyethylene (HDPE), AC, CI and DI with PVC, HDPE, GI and AC constituting over 70 percent of total pipe network for all water utilities. In this regard recorded pipe burst rates are mostly higher targeted thresholds leading to high water losses (UN-Habitat, 2011a; UN-Habitat, 2011b; UN-Habitat, 2011c; UN-Habitat, 2011d). The threshold for SRWB is 0.55bursts/km/year (SRWB, 2015) but up to 7bursts/km/year actual burst rate has been recorded in some cases (SRWB, 2014). Similarly BWB and LWB have burst rate thresholds of 0.9bursts/km/year (BWB, 2015) and 0.3bursts/km/year (LWB, 2014) respectively.

2.3 Leakage Management as a Water Demand Management Strategy

The available water sources throughout the world are getting depleted. This problem is further aggravated by climatic change and the rate at which populations are increasing especially in developing countries. Urban settlements in developing countries are at present growing five times as fast as those in developed countries. Between year 2000 and year 2030, urban population is expected to grow from 1.9 billion to 3.9billion, averaging 2.3% per annum (Dighade, Kadu, & Pande, 2014).

The urban population in Malawi is projected to grow by an average growth rate of 4.3% between 2012 and 2033 (NSO, 2009a). This is expected to exert pressure on available water resources for urban population while further straining meager financial resources for capital investments in urban water supply infrastructure for expansion and rehabilitation programs. Malawi has a number of features of water systems and the economy which make water conservation through water demand management imperative. The first is that although surface water availability seems to be adequate, most of it occurs as surface runoff during the wet season, so it is not available for human consumption or agriculture. The second is that due to the seasonal nature of rainfall, water is abundant during wet season but scarce during dry season. The third feature is the uneven distribution of water resources, implying that some parts of the country are relatively drier that others (Mulwafu, et al., 2002).

Mulwafu, et al. (2002) also observed that water resources in Malawi have come under increasing pressure from rapid population growth and economic activities such as farming, industrialization and urbanisation. The use of water in these activities has compromised water quality. They also envisaged that intensification of these activities in future will increase demand for water and cause threats to the availability and quality of water.

In 2008, Lilongwe Water Board was able supply 69,000m^3 per day against a demand of 73,250m^3 (UN-Habitat, 2011b); Blantyre was able to supply 79,410m^3 per day against a demand of 96,000m^3 per day (UN-Habitat, 2011a); in Mzuzu city the Northern Region Water Board was only able to carter for 60 percent of water demand (UN-Habitat, 2011c) while in Zomba city, the Southern Region Water Board was able to meet 100 percent water demand of its 145,000 people per day (UN-Habitat, 2011d). This failure to meet demand is against a background of very high distribution water losses of 44 percent for both Lilongwe and Mzuzu cities. If the two cities had invested and focused on water demand strategies in form of leakage control and reduce the distribution losses by at least 40 percent, they could have met 100 percent of their daily water demands (UN-Habitat, 2011c).

In South Africa, the Water Research Commission (WRC) has concentrated on providing low cost software solutions to help water utilities in managing their unaccounted-for water as part of water demand management tools in resolving water resource scarcity. This is in light of the fact that South Africa is one of 20 countries in the world that are highly water stressed (McKenzie and Bhagwan, 2000). The Water Research Commission has also indicated that the requirement for water in South Africa has been growing at between 4 and 5 percent since the 1930s and therefore predicts that if demand for water continues to increase at this rate, many parts of that country will effectively enter a state of continuous water stress within the next 50 years (McKenzie and Bhagwan, 2000). Consequently, the WRC is championing the emphasis from the purely supply oriented approach to one of both supply and demand management with the latter receiving priority.

Therefore, efficient management of meager water resources at every stage in the production and distribution systems would ensure minimal water wastage and losses hence benefiting both the consumer in terms of accessing right quantities at reasonable cost and the utility company in terms of service reliability and optimal running costs. Pressure management in water distribution systems leads to reduction in pipe and valve failures which ultimately result in reduction in physical water losses. Pressure management is therefore used as a tool to achieve water demand objectives.

2.4 Water Loss

As defined in section 2.1 above, water distribution system is a network of pipes, pipe nodes, water tanks, pumps and valves (Giorgio-Bort, Righetti, & Bertola, 2014). In some cases, particularly commercial water utilities, WDS also comprise water meters. The purpose of a WDS is to extract, sometimes treat (purify), transmit, store and distribute water to users (often referred as consumers). In the process of extracting, transmitting and distributing water to the consumers, it is always found that there is a difference between water entering and leaving the conveyance system. This difference is known as water loss or Unaccounted for water (UFW). The loss is determined by a comparison or difference between water passing through a flow meter from a source or water treatment plant (WTP) and an aggregate of flow readings from district meters (also known as Area meters) or consumer meters. The terms Unaccounted for Water and Non-Revenue Water (NRW) are often interchangeably used to mean water loss in a WDS despite having some distinction between them. Sharma (2008) describes UFW as representing the difference between "net production" (the volume of water that can be accounted for by legitimate consumption, whether metered or not) and legitimate water consumption. Thus:

UFW = net production – legitimate consumption.

NRW represents the difference between the volumes of water delivered into the water distribution network (WDN) and billed authorized consumption. Thus:

NRW = Net production – Revenue water

= UFW + water which is accounted for, but no revenue is collected (unbilled authorised consumption)

Non-Revenue Water is expressed as a percentage of net water produced (delivered to the distribution system or as m^3/day/km of water distribution pipe system network (specific loss), m^3/day/connection or m^3/day/connection/m pressure. Water loss as percentage of net water production is the most common (Sharma, 2008).

There are two types of water losses; physical water losses (also known as real losses) and apparent losses. Examples of physical losses are leakage through loose pipe joints, tank overflows, leaking tank walls and loose leaking valves. Examples of apparent losses include invisible losses through water meter reading errors, water meter under-registration, and water theft and authorised unmetered water consumption.

Reduction of physical water losses being the objective of this study, more emphasis has been put on previous research on physical water losses.

2.4.1 Physical Losses in Water Distribution Systems

2.4.1.1 *Leakage Control Approaches for Physical Water Loss Reduction*

Several studies have been conducted over the years in this area. Xu, et al. (2014) carried out a comprehensive review on the potential water leakage control approaches in water distribution networks focusing on environmental benefits. They used the following instruments to achieve their objectives; improvement of leakage by combining models and instruments, pipe repair, rehabilitation and replacement, and water pressure regulation. In the three methods above, Xu, et al. (2014) observed one setback being laborious, tiresome and unsustainable. This is true considering the physical mobility and the odd hours at night required by the observers to undertake the work which poses a number of health and security risks. Xu, et al. (2014) also made an important observation that most studies only consider pipe repair and pipe replacement to develop optimal pipe maintenance models because of the difficulty in the prediction of pipe break rate after rehabilitation.

Giorgio-Bortet, al. (2014) undertook a study on methodology for leakage isolation using pressure sensitivity and correlation analysis in water distribution systems. The study focused on performing an analysis of the sensitivity and correlation of the network with the objective to extract the best measurement points (i.e., nodes). This is described as a new leakage localization that is calibrated on real networks. Leakage positions are usually inferred by measuring pressure in a certain number of nodes of the network. Giorgio-Bort, et al. (2014) observed that by comparing data collected from flow and pressure meters, with the predictions coming from the simulations, it is possible to identify the entity of leakage and most probable area of the network in which the leakage can be found. Giorgio-Bort, et al. (2014) concluded that efficiency of leakage identification procedure is particularly sensitive to the quality of information available from the real network.

2.4.1.2 *Water Pressure Regulation for Physical Water Loss Reduction*

Water pressure regulation is an effective and efficient way to reduce leakage of water distribution systems and it is the only way to reduce background leakage that cannot be detected using current techniques (Xu, Liu, Chen, & Li, 2014). This is backed by the fact that leakage is positively related to pressure hence reducing water pressure immediately reduces leakage (Lambert, 2001; Thornton, 2003;Thornton and Lambert, 2007). Compared to long-term break detection and pipe maintenance strategy, water pressure management extends pipe's lifespan (Farley and Trow, 2003; Thornton and Lambert, 2006; Thornton and Lambert, 2011). Pressure management is achieved often by way of partitioning a water reticulation network into Pressure Management Area (PMA) and District Metering Area (DMA) where pressure reducing valves (PRVs) are installed at entry of a PMA or DMA. Flow meters are installed at entry and exit of a DMA to record variations in NRW. Water pressure in PMA is regulated by a PRV. The challenge with PMA is that PRVs are so costly for most water utilities in developing countries to afford or sustain while DMA is laborious and demands working at odd hours such as from midnight to early morning hours to observe minimum night flows, hence posing physical and security risks, and inconvenient for field data collection participants. In this regard, Araujo, et al. (2006) developed a model that optimises the installation and operation strategies for PRVs that includes the quantity, locations and opening adjustment to do away with challenges just outlined above. The setback with this model however, is that, developing countries hardly use it due to lack of decision support tools that can accurately assess the benefits associated with pressure management to justify the investment. In addition to that, it is an established fact that water distribution networks are usually not well configured for effective pressure management (Mutikanga, Sharma, & Vairavamoorthy, 2013). The two observations apply to the current state of Thyolo Boma WDS. There are no proper pressure management programs in place and the WDS is not configured.

2.4.1.3 *Challenges in Water Loss Management of WDS in Developing Countries*

Dighade, Kudu and Pande (2014) studied challenges in water loss management in developing countries. The objective was to provide a comprehensive insight of issues pertaining to the challenges in water loss management of water distribution systems in developing countries with particular focus on saving water through water loss control/reduction to benefit fast growing urban population in developing countries in light of other pressures on water resources such as climate change and depleting water resources due to competing water uses. Dighade, et al. (2014) described leakage as being not only an economic issue as often perceived and presented by water utility companies and institutions but also an environmental sustainability and potentially a health and safety issue (Colombo & Karney, 2002).

The condition of the infrastructure and the renewal or rehabilitation policy in water distribution systems is probably one of the main reasons for variation in leakage across the world. This is more pronounced in developing countries with ageing infrastructure (Kingdom, Liemberger, & Marin, 2006). Dighade, et al. (2014) argues that high levels of real loss reduces the amount of precious water reaching customers, increases operating costs of the utility and makes capital investments in new resource schemes larger. To this effect, they carried out a comparison of water losses between developed and developing countries and observed that current statistical surveys indicate that NRW in developing countries is around 45 to 50%. However, they made a contradiction that despite such statistics, the fact is, very few data are available in literature regarding actual figures. This was attributed largely to the fact that most water utilities in developing countries lack national reporting system of their WDS. This is true of Malawi. The country has only begun a national reporting system in the 2015/2016 financial year through Water Services Association of Malawi (WASAMA).WASAMA has initiated collection and consolidation of information on water utility performance (Benchmarking) after learning from both selected developed and developing countries such as Canada, Portugal, Netherlands, Mexico, Ethiopia, Kenya and South Africa on the importance and the benefits of benchmarking.

Dighade, et al. (2014) also pointed out that Municipal water pipe distribution networks deteriorate naturally over a period of time and subsequently leakages occur in the pipe network citing the following issues as factors causing high NRW in developing countries:

1. Deterioration is caused by corrosive environment, soil movement, poor construction practices and workmanship, fluctuation of water pressure and excessive traffic loads and vibration.
2. Water utility managers in developing countries invariably face greater challenges including the rapid urbanisation, diminishing water resources, intermittent water supply systems with very poor supply hours, increased NRW, very poor system pressure, inequitable distribution

of the available water, outdated and poor infrastructure, poor operations and maintenance policy, poor record keeping systems, inadequate technical skills and technology,

Dighade, et al. (2014) therefore came up with three diagnostic approach strategies for water loss management applicable to any WDS in developing countries as solutions to challenges outlined above that included water audit, performance indicators and benchmarking, and pressure management

In summary, municipal water distribution pipe network deteriorate naturally over a period of time and consequently leakage occurs. This deterioration is caused by three man factors categorised as physical, environmental, and operational. Leakage is directly proportional to pressure in a WDS hence pipe bursts are a function of pressure. Pressure management, leak detection and control programs bring about substantial physical water loss reductions in a water distribution system. High population growth rates and rapid urbanisation in Africa, and Malawian cities and towns in particular, are exerting pressure on limited water resources leading to failure by water utilities to meet fast growing water demand. In this regard, pressure and leakage management have proved to be useful strategies in reducing water demand gap since the water resources saved in form of physical water loss reduction are channeled to meeting new water demands without necessarily expanding water supply infrastructure.

Over the years, hydraulic models to improve management and operational efficiency of WDSs have been developed and improved to ease solving complex WDS analyses. Apparently, financial limitations for some countries and water utilities particularly in developing countries, remains a major setback in acquisition of certain improved technologies, failure to implement WDS rehabilitation plans and development of robust water supply networks which make their WDSs unreliable with high levels of physical water losses. Literature has also revealed that continuous WDS rehabilitation and upgrading programs that are often delayed or ignored in developing countries including Malawi, are very crucial in controlling physical water losses through renewal of deteriorating supply infrastructure that would otherwise be the source of pipe bursts and leakage.

CHAPTER 3: RESEARCH METHODOLOGY

3.1 Introduction

The aim of the study as presented in Chapter One was to characterise the hydraulic performance of the existing water distribution system for Thyolo Boma in order to find solutions that would be recommended to SRWB management for improvement of physical water losses. This would be achieved through evaluation of current hydraulic regimes of in the distribution system to see if they are within permissible ranges for effective operational efficiency or outside permissible ranges that may be causing pipe failures and hence high non-revenue water, EPANET hydraulic analysis results would be used to map the regimes that will depict and guide on the areas requiring attention hence solutions and recommendations worked proposed.

3.2 Hypotheses

The study was firstly based on three hypotheses by Thornton (2003): that rate of leakage in water distribution systems is a function of pressure that is applied by either pumps or gravity head and therefore leakage is directly proportional to pressure; that pipe burst is a function of pressure. Secondly, it was based on Bernoulli's total energy principle stating "the total energy possessed by a unit mass of water comprise; (i) its kinetic energy of movement; (ii) its pressure energy; and (iii) its position energy above some given datum in a gravitational field.

3.2.1 Conceptualisation

The hypotheses in 3.2 above stimulated ideas that the principles outlined were very much relating to the current state Thyolo Boma WDS is in. To that effect, the following concepts were formulated prompting the research:

- Thyolo Boma's varying topography could mean wide elevation differences between two relative points in the water distribution system that subsequently give rise to the position energy in the Bernoulli's energy principle.
- High position energy would mean high gravity heads leading to high pressure energy (dynamic or operating pressure) in the pipe network.

- The high pressure energy would give rise to high kinetic energy as water begins to flow in the WDS that would consequently give rise to pipe burst and leakage since pipe bursts and leakage are a function of pressure.
- Deteriorating WDS infrastructure such as aged pipes are prone to damage hence increased leakage and pipe bursts.

On the basis of these four points, it was conceived that pressure management on Thyolo Boma WDS using EPANET to determine its hydraulic regimes (water pressure, flow velocity and unit head loss), would reduce physical water losses.

3.3 Research Design

3.3.1 Research Philosophy

The study adopted a combination of empirical and analytical research.

3.3.2 Research Strategy

This was a case study where the existing water distribution system for Thyolo Boma was evaluated by analysing its hydraulic regimes using EPANET, water distribution design software, and the results correlated with historical data on pipe failure/leakage and the factors.

3.3.3 Reliability and Validity

Consistency in results analysed from a desk study on SRWB's pipe-break analysis from historical records and water distribution simulation results (pressure pattern) were compared to ensure reliability. Basic mathematical calculations were used to analyse and compare relevant desk study data and some model results for interpretation.

A section of one pipeline, identified as vulnerable, with high frequency of pipe failure and high distribution pressures has had its deteriorated AC pipes replaced with new PVC pipes with suitable pressure rating for observation as a control to validate the results. Actual pressure measurements were taken at selected points along the distribution system (see Results) and compared with simulated pressure results to establish variations and errors if any and hence validate the simulated results.

3.4 Methodology

In order to find answers to the research questions, a procedural framework within which the research was to be conducted was developed. In this case, the research was carried out through desk and field studies. The following were steps and procedures taken to arrive at answers to the research questions.

3.4.1 Desk Study

The desk study was carried out basically to appraise the researcher on existing literature on what has been researched before on similar topics raised in research questions and specific objectives. Historical performance records for the existing Thyolo Boma WDS were reviewed and analysed. These were carried out in order to effectively decide what information was required; how it could be collected; what factors were relevant and how it could be used. The desk study was also carried out prior to field study because; that would help identify materials and tools pertinent to the areas under investigation; lessons were to be learnt from what other researchers had done in the same fields being studied; it would help stimulate questions during analysis and broaden the researcher's perspectives and set the work in context; it would provide alternative explanation to the questions; it would complement, extend and verify findings; would help identify appropriate methodology to employ, spot areas not yet researched or requiring further research as well as propel ideas for the study.

3.4.1.1 *Sources of Information for the Desk Study*

The literature reviewed during the desk study was mainly from the following sources:

- Journals and Conference papers from various individuals and institutions with authority in population and socio-economic issues, water resources, water supply and environmental management including United Nation bodies and the World Bank websites.
- Urban Profiles for Lilongwe, Blantyre, Mzuzu, and Zomba city councils.
- Strategic Plans for Blantyre Water Board and Southern Region Water Board
- Corporate Charters (Service Level Agreements) between Government of Malawi (the Shareholder) and individual public water utilities, SRWB and LWB.
- Annual Performance Reports for SRWB and BWB.
- Polytechnic Library, University of Malawi.
- Past MSc and PhD Theses.

- Annual and quarterly performance evaluation reports for SRWB including Faults Registers at local customer interface office at Thyolo.
- Discussions with academic project supervisor and senior and middle management at SRWB including a few selected artisans and technicians directly involved in managing Thyolo WDS.
- Distribution network maps and drawings for Thyolo including data on pipe age (date installed), type and size.
- Extraction of maximum consumption data from customer water accounts from the SRWB's billing system.

3.4.2 Field Study

The problem research questions required a hydraulic performance analysis of the existing water distribution system for Thyolo Boma. This required hydraulic modelling using EPANET. In this regard, input data for EPANET required both field study data obtained through topographic survey, some water distribution infrastructure measurements and desk study in the form of base demand data from SRWB billing/accounting system, water reticulation layout and specifications for key structures like pumps, pipes and tanks.

3.4.3 Data Collection and Analysis

In order to find answers to the research questions that are aimed at addressing the specific objectives of this study, the following steps and procedures were followed.

3.4.3.1 *Linkage between Pressure Distribution Pattern and Rate of Pipe Bursts*

The first research question was about how pressure distribution pattern relate to the rate of pipe bursts. To arrive at the answer for this question, the following procedure was taken.

(i) Literature on previous studies on the relationship between water distribution pressure and pipe breakage as well as leakage was reviewed during the desk study.

(ii) Pipe burst analysis for a three year period from 2011 to 2014 to get a fair annual average for individual pipelines was carried out during the desk study. A pipe burst rate expressed as 'bursts per kilometre per year' (bursts/km/year) was determined for each pipeline for comparison with SRWB's standard of 0.55bursts/km/year.

(iii) Base demand for each key node on the water reticulation network was determined by extracting water consumption data from SRWB's billing system for all customer accounts in an area supplied from that node. A factor of 1.44, representing the 44 percent water loss in Thyolo Boma WDS as indicated in the problem statement, was then applied to the aggregated consumption to determine base demand as one of the required EPANET input data.

(iv) Using a GPS, a topographic survey to collect spot elevations and coordinates for key WDS structures such as pipe junctions, pipe transition points (pipe reducers, pipe type change points), reservoirs, storage tanks and pumping stations, was carried out. Elevations were required as EPANET input data while elevations were necessary in case of trying to locate a point to confirm data for one reason or another.

(v) Where records were not available or not clear, pipeline distances, tank heights and diameters, tank levels (initial, minimum and maximum) re-measured.

(vi) Data on pipe type, nominal diameter (internal diameter), and year of installation (to determine age), were collected and confirmed from water reticulation maps and other official records. These were used as EPANET input data.

(vii) Pipe friction factors for each pipe material type, were obtained from SRWB standard WDS criteria and Arnalich (2011) EPANET learner's manual for use as EPANET input data.

(viii) Data on pump specifications such as pumping head, power rating, and flow rate required as input data in EPANET was collected and confirmed.

(ix) Where storage tanks of a shape other than circular, an equivalent diameter was calculated and used as input data in EPANET.

(x) A schematic water distribution system was drawn on EPANET workspace.

(xi) Input data was entered in relevant EPANET files and thereafter analysis was run.

(xii) When the analysis was successful, results were viewed and extracted. In the case of this research question, a pressure map was viewed and results were compared with pipe burst analysis results to establish a link between pressure head magnitudes and burst rates.

(xiii) The results were therefore interpreted by comparative analysis. The collected data was mainly quantitative

3.4.3.2 *Linkage between Elevation, Flow Velocity and Pipe Burst*

This question was trying to find out if elevation and flow velocity at a particular point in a distribution system contribute to pipe burst. In order to get answers, Permissible flow velocities obtained through literature review during the desk study were compared with those from EPANET model results to establish linkage between velocity variations to pipe burst rates. A trend of the level of an elevation against pipe burst rate was established and the details are outlined in chapter 4 (Results and Discussions). The results were therefore interpreted by comparative analysis. The collected data was mainly quantitative

3.4.3.3 *Age versus Pipe Bursts*

This question was investigating the relationship between pipe age and burst rate. The following procedure was followed:

(i) A schedule of pipe installation records showing number of years of service was prepared from Thyolo Boma water reticulation maps and other technical records during the desk study

(ii) The pipe age records were compared with both pressure distribution pattern from EPANET model results and pipe burst rates to establish any trend linking levels of pipe breakage with respect to age. The results were therefore interpreted by comparative analysis. The collected data was mainly quantitative. Details of that outcome are outlined in chapter 4 under Results and Discussions.

3.4.3.4 *Pipe Characterisation versus Pipe Bursts*

The question was about whether pipe size, type, nominal pressure (pressure rating) and corrosion relate to frequency of pipe bursts. To get answers, the following procedure was followed.

(i) From water reticulation maps and other technical records, a schedule of pipe type (named by material type), pipe size (diameter) and nominal pressure (pipe pressure rating) was prepared.

(ii) Where galvanised iron, ductile cast iron and cast iron pipes are installed, a trench to expose at least one full length of pipe on a pipeline was dug to sample the corrosion state.

(iii) The data for variable in (i) and the corrosion state of a pipe in (ii) above were compared pressure pattern from EPANET model results and pipe burst rate results to establish linkage of the variables. See detailed results chapter 4.

(iv) The results were therefore interpreted by comparative analysis. The collected data was mainly quantitative.

3.4.3.5 *Hydraulic Modelling with EPANET*

Hydraulic modelling is a simulation of fluid flow and evaluation of its important elements. Hydraulic modelling can generally refer to both numeric modelling (in which simulation is done on a computer), or physical modelling (where the physical flow geometry is scaled in such a way that it can be modelled in the laboratory).

EPANET is a computer program that performs extended period simulation of hydraulic and water quality behaviour within pressurised pipe networks. A network consists of pipes, nodes (pipe junctions), pumps, valves and storage tanks or reservoirs. EPANET tracks the flow of water in each pipe, the pressure at each node, the height of water in each tank, and the concentration of a chemical species throughout the network during a simulation period comprised of multiple time steps. In addition to chemical species, water age and source tracing can also be simulated (Rossman, 2000). EPANET is designed to be a research tool for improving understanding of the movement and fate of drinking water constituents within distribution systems. It can be used for many different kinds of applications in distribution systems analysis. Sampling program design, hydraulic model calibration, chlorine residual analysis, and consumer exposure assessment are some examples. EPANET can help assess alternative management strategies for improving water quality throughout a system. These can include:

- altering source utilization within multiple source systems;
- altering pumping and tank filling/emptying schedules;
- use of satellite treatment, such as re-chlorination at storage tanks;
- targeted pipe cleaning and replacement.

Running under Windows, EPANET provides an integrated environment for editing, network input data, running hydraulic and water quality simulations, and viewing the results in a variety of formats. These include colour-coded network maps, data tables, time series graphs, and contour plots. Input data includes schematic representation of key parts of the actual WDS (water source, pumps, pipes, tanks, valves and pipe junctions) on EPANET workspace, elevations for junctions (nodes), pipe diameter, pipe length, pipe friction factor, pump head, pump power rating, valve type and size, tank diameter, tank water levels (initial, minimum and maximum), base water demand for nodes etc. Output data or results include pressure, head, unit head loss, flow rates, flow velocities etc. The model requires making particular option settings like types of hydraulic analysis formula to use, units of measure, variable pump settings if any, and single or extended simulation.

In order to determine hydraulic regimes (e.g. water pressure, flow velocity and unit head loss) in the distribution system of the study area, the hydraulic modelling of the system with EPANET software was carried out in this study.

3.4.3.6 *EPANET Pressure Validation*

To authenticate the EPANET model results, the study made a provision to take actual measurements of the water distribution system's operating pressures and compare them with modelled pressures. Pressure was chosen among the variables under investigation because it was thought to be the main factor causing pipe failure therefore deserving more attention.
The following procedure was taken to validate the results.

(i) Readings were taken at designated times targeting off-peak and peak water demand periods of the day for two days near nodes with either very high or low pressures from the pressure map (see Results, Figure 9). The challenge however was that readings for different pipelines were taken on different days due to limitations in tools, human, and material resources. The other reason was to minimise interrupting water supply to consumers during installation of pressure gauges.
(ii) Two pipelines were clamped at a time. This might have a bearing on consistency of measured pressure results because tank water levels fluctuate on daily basis for a number of factors such as maintenance in the WDS or intermittent water production resulting from power outages.
(iii) The highest pressure reading measured in (i) above for each point was adopted.
(iv) The results were compared with model results for the same spot (node).

3.4.3.7 *Optimisation of Water Distribution System*

This study focused on proposals that are more or less of operational and rehabilitation of parts of problematic areas in terms of excessive pressures and high frequency of pipe bursts leading to high water losses. The following procedure was taken to ensure optimisation of key elements of the WDS.

(i) **Reinstatement of two storage tanks at Glennae and Boma**. Initially the two areas had an elevated pressed steel tank each but later on abandoned due to excessive leakage from worn out tank panels. The tanks were therefore bypassed resulting in rise in pipe bursts and leakage areas downstream. It was suspected that this was because the areas were now being fed from Number One main service reservoir which is at a much higher elevation than the two tanks. It was therefore decided that reinstating them would restore 'Break-Pressure' facilities from the main service reservoir and possibly normalise permissible dynamic pressures to the areas. This was simulated in EPANET until permissible pressures were obtained.

(ii) **Installation of Pressure Reducing Valves (PRVs)**.For pipelines with very high dynamic pressure and where installation of a service tank as Break-Pressure facility would require new costly capital investment, it was proposed to install PRVs to reduce and regulate pressure downstream from a selected strategic node in terms of pressure transition (low to high) along that distribution pipeline.

(iii)**Change of pipe size or pipe type**. Through EPANET, several pipe size iterations would be made and monitor where, change of pipe size, with just very few pipes required in bringing high and low pressures to permissible levels would be undertaken.

(iv)**Installation and operation of water distribution infrastructure**. Sometimes pipe failures and leakage emanate from the way structures are constructed (poor workmanship) like installation of plastic and asbestos cement pipes in either shallow pipe trenches or pipes are left exposed above ground for one reason or another and how certain facilities are operated for instance rapid opening/closing of valves when the rule is to open/close gradually thereby inducing transient pressures (pressure surge). These are some of the factors that cause pipe breakage. Where such instances are observed during the field study, recommendations for redress shall be made.

3.5 Summary

Literature review played a very important role in having an insight to designing the research methodology particularly the research concepts and theoretical framework. Similarly, the availability of a water distribution system planning, design and simulation model, EPANET, eased the otherwise complex statistical and mathematical that could have been undertaken in its absence. EPANET modelling procedures as required by the software also guided most steps and procedures outlined to get answers to the research questions and hence the research design.

CHAPTER 4: RESULTS AND DISCUSSION

4.1 Introduction

In this chapter, output data from EPANET analysis for review of existing Thyolo Boma WDS, and pipe burst data from SRWB Thyolo office faults register are analysed. The collected data was mainly quantitative; the results were therefore interpreted by comparative analysis.

To validate these results, actual pressures were measured using pressure gauges clamped to selected pipelines particularly those showing very high pressures then compared with those obtained from EPANET results. The existing WDS was further manipulated in the EPANET with the purpose of obtaining an ideal or near ideal optimal hydraulic regime that would bring them within permissible ranges so as to resolve challenges being addressed by the study's objectives.

4.2 Data Analysis and Discussion

This section analyses all results from processed raw data obtained from field and desk studies. As earlier indicated under methodology in chapter three, both raw and processed data is generally quantitative, therefore interpretation of results will be comparative analysis.

4.2.1 EPANET Results from Existing WDS

4.2.1.1 *Pressure Variation*

EPANET analysis results gave pressures ranging from 9.42m to 164.11m (refer to Tables 2 and 3) at pipeline junctions (nodes) and zero to 0.7m at reservoirs (sources) and tanks respectively. SRWB considers 10m manometric head as minimum for both gravity fed and pumping mains and 70m as maximum operating heads for gravity fed mains in its Water Distribution Systems (SRWB, 2009). For pumping mains, the guiding principle is that operating pressure head must be 80% or less of the pipe's nominal pressure rating so that water is lifted to the required head without bursting the pipe (SRWB, 2009). However, minimum and maximum distribution pressures of 10m and 30m respectively are recommended for all other (particularly gravity-flow) water distribution pipelines (Arnalich, 2011).

In the case of Thyolo WDS results, the following areas were critical:

I. Pumping main from Nsuwazi WTP to Boma at Number One had the highest pressure at 164.1m close to the pumping station (node 22) and it declines as it nears the Break-pressure tank/ service reservoir at Number One (Nodes 22 to 29).

II. Number One – Kasembereka gravity fed distribution pipeline had the highest pressures ranging from 92m to 162m (Nodes 85 to 91).

III. Number One – Ndalama gravity fed line had pressure range of 78m to 87m (Nodes 103 and 111)

IV. Boma area gravity fed pipelines had pressures ranging from 43m to 69m (Nodes 34 to 134).

V. DC's lines – Agriculture/Nachipere pipeline had pressures ranging from 79m to 85m (Nodes 74 to 136).

VI. Nchilamwera – Kalilombe – Nchenachena – Nachipere gravity fed pipeline had pressures ranging from 74m to 118m (Nodes 55 to 68).

VII. Boma – Glennae gravity distribution pipeline had pressures ranging from 83m to 141m (Nodes 113 to 138).

VIII. Mpeni WTP – Number One BPT gravity fed transmission pipeline had pressures ranging from 22m to 53m (Nodes 9 to 29 plus 99).

IX. Mpeni intake - Mpeni WTP pumping main had pressures ranging from 13m to 95m (Nodes 3 to 8).

Figure 6 below is a map depicting pressure variation from the analysis of the existing water distribution system.

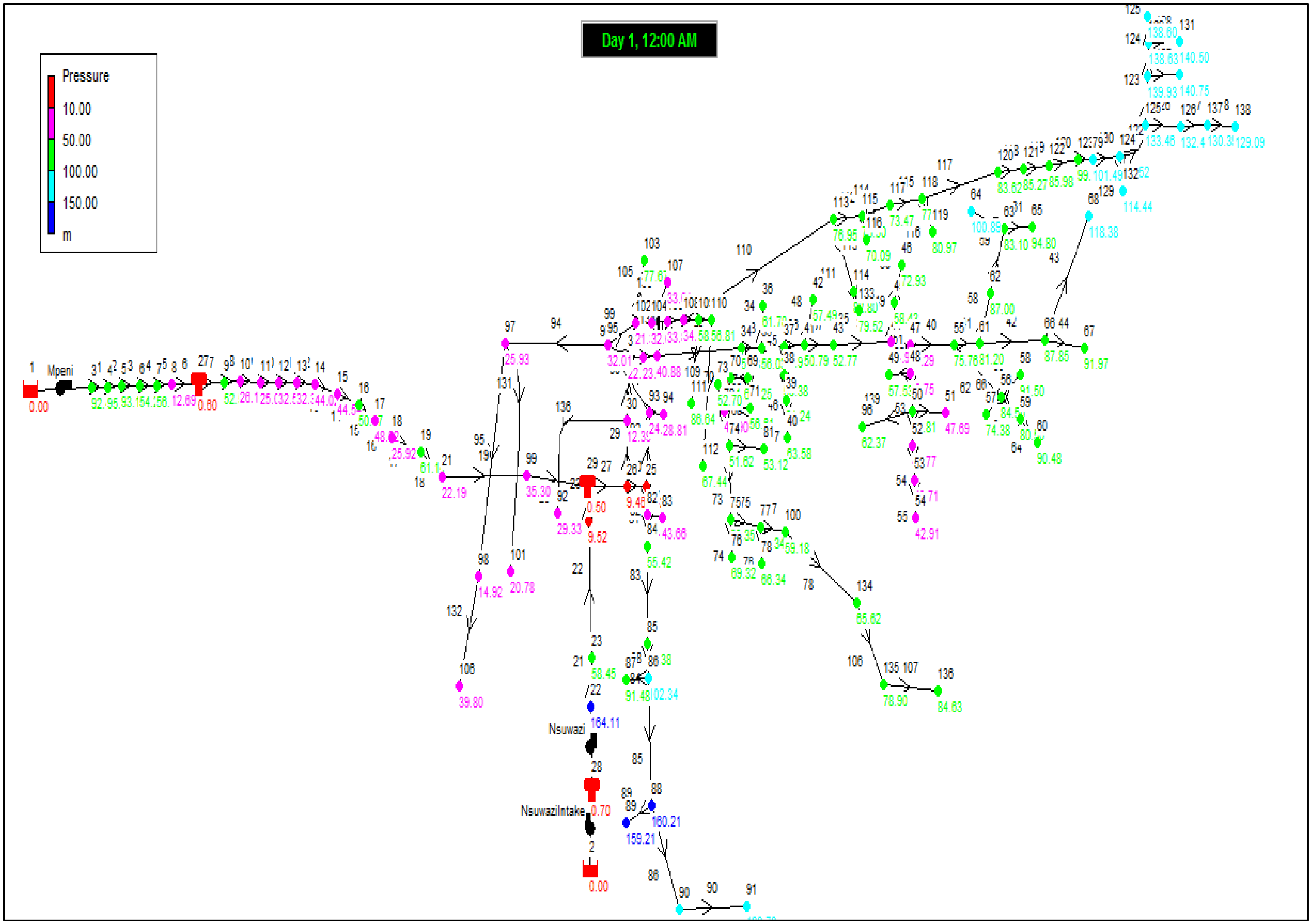

Figure 6: Pressure (m) Variation Map for Existing Thyolo Boma WDS

Based on SRWB operating distribution pressure design criteria that prescribe permissible working pressure range of 10m to 70m, only two areas, Boma area distribution lines, and Mpeni WTP – Number One, with colour codes mainly pink and green on the pressure map, were the ones within permissible pressure range. The rest, with the exception of the two pumping lines, had above the range pressures. This can be attributed, as shown by the pressure variation map in Figure 5, to the fact that the pipelines with permissible pressure are located in higher elevation areas where as those with above permissible pressures are located in low to very low elevation areas hence being in agreement with Bernoulli's potential energy principle. Arnalich (2011) points out that it is very important that water supply systems function at the smallest possible pressures over and above the design range. When systems carry a lot of pressure, the leaks are greater and therefore pipes break more often. This is indeed reflected on the ground as can be seen on the pipe-burst analysis where pipelines in high elevation areas have the least failure rate because low potential (gravity head) energy in contrast to pipelines in low elevation areas with higher potential energy that have higher failure rates and hence high leakage. In practice, pressures as low as 5m are still adequate for most users. The only challenge is that such pressure can not suffice municipal firefighting requirements (SRWB, 2009). The model pressure results correlated very well with validation pressure results albeit minor variance of up to 5.7 percent with EPANET model results being generally higher than validation results (Figure 9). Detailed pressure results for selected nodes for existing WDS are scheduled in Tables 2 and 3 below.

Table 2: Least Pressure Pipe Network Junctions and their Elevations from Existing WDS EPANET Analysis Results

Network Table - Nodes

Node ID	Elevation m	Head m	Pressure m
Resvr 2	841	841.00	0.00
Resvr 1	998	998.00	0.00
Tank 29	1005	1005.50	0.50
Tank 27	1095	1095.60	0.60
Tank 28	845	845.70	0.70
Junc 25	996	1005.42	9.42
Junc 26	996	1005.46	9.46
Junc 24	996	1005.52	9.52
Junc 30	993	1005.39	12.39
Junc 8	1083	1095.69	12.69
Junc 98	990	1004.92	14.92
Junc 82	985	1004.66	19.66
Junc 101	982	1002.78	20.78
Junc 102	983	1004.39	21.39
Junc 31	983	1005.02	22.02
Junc 21	1014	1036.19	22.19
Junc 32	981	1004.87	23.87
Junc 93	981	1005.11	24.11
Junc 11	1069	1094.02	25.02
Junc 18	1057	1082.92	25.92
Junc 97	979	1004.93	25.93
Junc 10	1068	1094.15	26.15

Table 3: Highest Pressure Network Junctions from Existing WDS EPANET Analysis Results

Network Table - Nodes

Node ID	Elevation m	Head m	Pressure m
Junc 85	909	1004.38	95.38
Junc 123	888	987.51	99.51
Junc 64	885	985.89	100.89
Junc 79	886	987.49	101.49
Junc 86	902	1004.34	102.34
Junc 132	865	979.44	114.44
Junc 68	866	984.38	118.38
Junc 138	856	985.09	129.09
Junc 124	856	985.52	129.52
Junc 137	855	985.39	130.39
Junc 91	869	999.70	130.70
Junc 126	853	985.44	132.44
Junc 125	852	985.46	133.46
Junc 130	844	982.60	138.60
Junc 129	844	982.63	138.63
Junc 127	844	983.93	139.93
Junc 131	842	982.50	140.50
Junc 128	843	983.75	140.75
Junc 90	854	1000.09	146.09
Junc 89	845	1004.21	159.21
Junc 88	844	1004.21	160.21
Junc 22	847	1011.11	164.11

4.2.1.2 *Flow Velocity*

Flow velocities, as shown below in Table 4, Table 5 and Figure 7, ranged from 0 m/s to 1.65m/s. From Tables 4 and 5, it can be observed that pipes with zero to less than 0.1m/s velocities (links 133, 87, 76, 104, 89, 103, 95, and 126) had low flow rates and low unit head loss with average pipe diameter of 100mm. This means the pipes are over-sized. Those with the highest velocities had correspondingly the highest flow rates with very high unit head loss too. Literature review (Table 1) indicates that rate of internal corrosion is greater in unlined dead-ended mains, which means that low flows in unlined pipes trigger corrosion and it has also been learnt that corrosion leads to leakage and pipe failure (FCMNRC, 2003). In this regard, all galvanized iron pipes and cast iron pipes, being unlined and with flows less than 0.5m/s are highly likely to have corroded and leakage or burst rates along them could be emanating from corrosion.

Table 4:Pipes with the Least Velocities from EPANET Analysis Results from existing WDS

Network Table - Links

Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km
Pump Mpeni	#N/A	#N/A	7.63	0.00	-99.43
Pump NsuwaziIntake	#N/A	#N/A	34.43	0.00	-4.70
Pump Nsuwazi	#N/A	#N/A	6.08	0.00	-165.41
Pipe 87	600	45	0.00	0.00	0.00
Pipe 133	40	29	0.00	0.00	0.00
Pipe 76	195	102	0.17	0.02	0.01
Pipe 104	387	102	0.20	0.02	0.01
Pipe 89	60	100	0.26	0.03	0.03
Pipe 126	400	57	0.10	0.04	0.06
Pipe 103	430	102	0.45	0.06	0.05
Pipe 95	176	102	0.46	0.06	0.05
Pipe 85	1612	100	0.52	0.07	0.08
Pipe 137	300	45	0.10	0.07	0.18
Pipe 60	718	29	0.04	0.07	0.30
Pipe 65	180	45	0.14	0.09	0.31
Pipe 94	520	102	0.82	0.10	0.15
Pipe 74	205	102	0.85	0.10	0.16
Pipe 125	69	45	0.19	0.12	0.50
Pipe 132	238	45	0.19	0.12	0.51
Pipe 70	168	40	0.15	0.12	1.17
Pipe 96	480	45	0.21	0.13	0.63
Pipe 106	1462	57	0.34	0.13	0.49

Table 5:Pipes with the Highest Velocities from EPANET Analysis Results of existing WDS

Network Table - Links

Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km
Pipe 45	100	45	1.18	0.74	15.54
Pipe 33	90	148	12.87	0.75	3.93
Pipe 34	40	102	6.28	0.77	6.36
Pipe 51	92	50	1.52	0.77	27.68
Pipe 127	20	81	4.04	0.78	8.64
Pipe 39	78	81	4.11	0.80	8.93
Pipe 35	445	102	6.59	0.81	6.97
Pipe 123	149	81	4.44	0.86	10.29
Pipe 27	6	200	27.87	0.89	7.07
Pipe 61	655	29	0.61	0.92	38.62
Pipe 88	150	29	0.61	0.93	39.10
Pipe 71	100	25	0.48	0.98	96.53
Pipe 67	80	81	5.30	1.03	26.69
Pipe 32	1007	148	18.17	1.06	7.44
Pipe 69	120	75	4.74	1.07	22.56
Pipe 68	157	75	4.82	1.09	23.22
Pipe 129	60	21	0.44	1.26	101.36
Pipe 28	298	150	28.32	1.60	29.54
Pipe 18	2303	148	28.32	1.65	16.91
Pipe 19	1294	148	28.32	1.65	16.91
Pipe 17	460	148	28.32	1.65	16.91

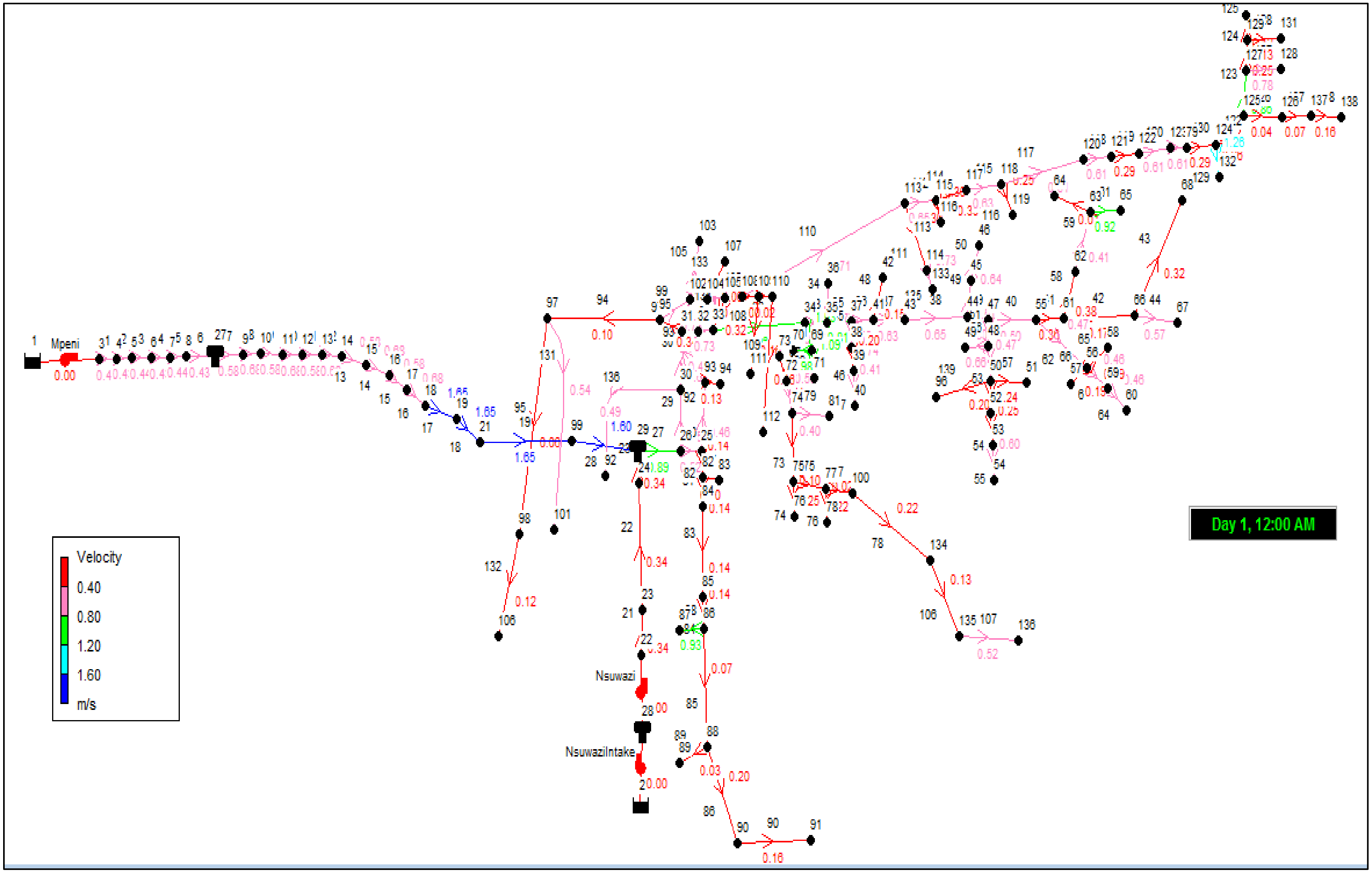

Figure 7: Velocity Variation Map for Existing Thyolo Boma WDS

Normal range of velocities at peak water demand time is from 0 (zero) to 2m/s for water without sediment. Velocities greater than 0.5m/s but equal or less than 2m/s are preferred in case of water with sediment to ensure that there is self-cleansing in pipes (Arnalich, 2011). From the continuity equation, $A_1V_1=A_2V_2$ (where A_1 and A_2 are pipe cross sectional areas at point 1 and 2 respectively and V_1 and V_2 are flow velocities at point 1 and 2 respectively. Velocity greater than 2m/s indicate that the pipe is too small. Small pipe diameter rockets friction hence high frictional head loss. From this criterion, a maximum velocity of 1.65m/s means that none of the pipelines (Tables 4 and 5) were undersized. Literature review (Table 1) has indicted that small diameter pipes are more susceptible to longitudinal failure along pipe length (FCMNRC, 2003). In this case failure due to pipe size (small diameter) can be ruled out but high probability of failure from over-sizing of pipes due to corrosion as outlined above.

4.2.1.3 *Unit Head loss*

Unit head loss, also referred to as hydraulic gradient, is a rate at which hydraulic pressure head is lost per unit length of a pipeline. This parameter can be considered either positive or negative in terms of pressure management in water distribution systems. In the case of very high working pressures in a water distribution system or particular pipeline, high unit head losses are deliberately introduced by under sizing pipe diameter to increase pipe friction losses that ultimately reduce pressure head downstream (Arnalich, 2011) whereas in lean working pressure scenarios, high unit head loss does frustrate delivery of water supply to the desired point by reducing desired working pressure head to below minimum requirement.

Arnalich (2011) recommends unit head loss values less than 5m/km for pumping mains and 10m/km for gravity fed pipelines. Below is a unit head loss map (Figure 8) of the existing Thyolo water distribution system.

It can be observed from the unit head loss map and EPANET model results (Tables 6 and 7) that all two pumping mains (Reservoir 1 to Tank 27 and from Reservoir 2 to Tank 29) had unit head losses within the recommended range of 0 – 5m/km with the maximum recorded being 2.60m/km. It can also be observed that the rest of the pipes which are all gravity fed, had a wide range of unit head losses varying from zero to 101.36m/km which is quite excessive.

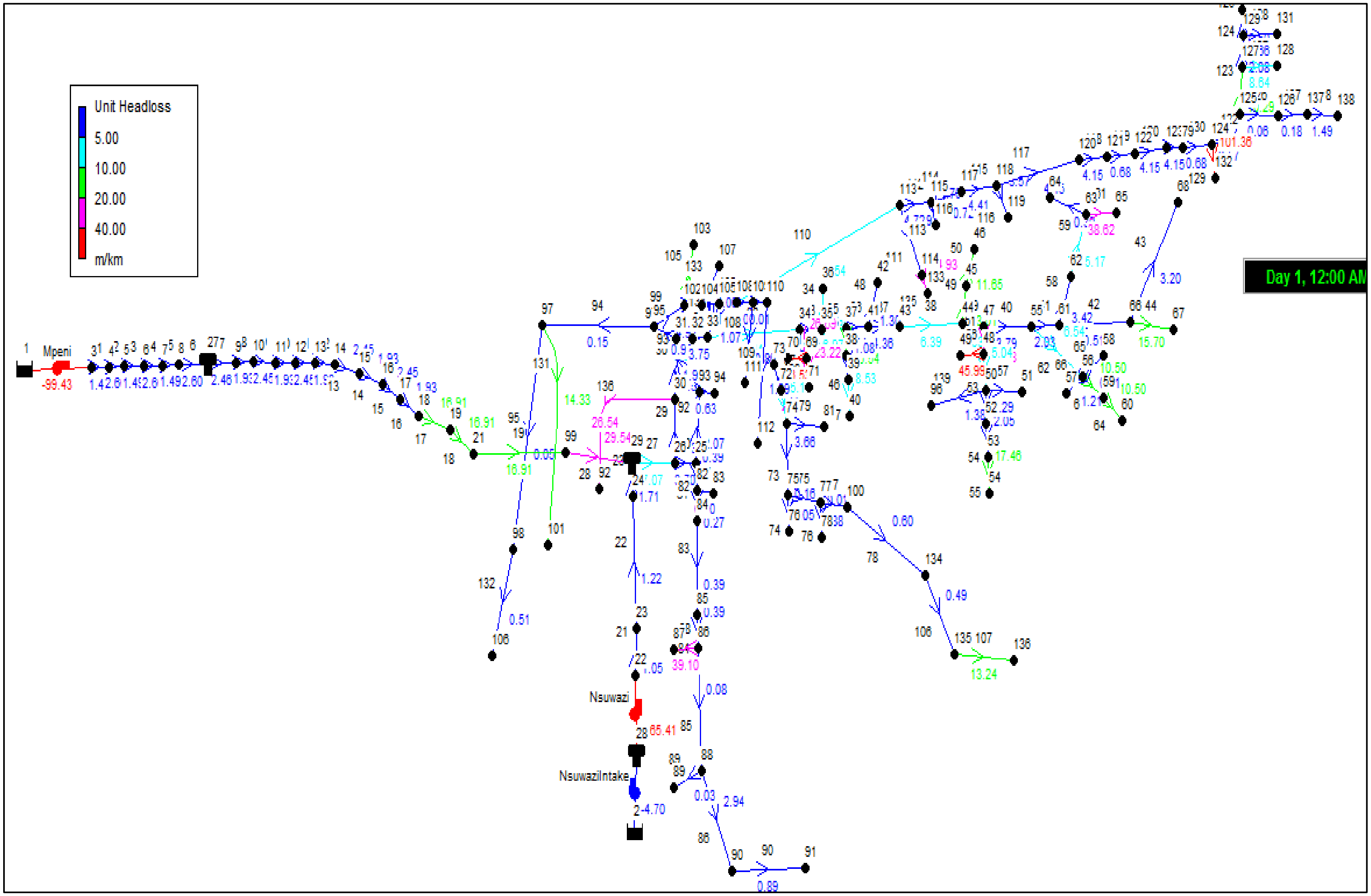

Figure 8: Unit Head Loss Map for Existing Thyolo Boma WDS

Table 6: Pipes with the Least Head Loss from EPANET Analysis Results of existing WDS

Network Table - Links

Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km
Pump Nsuwazi	#N/A	#N/A	6.08	0.00	-165.41
Pump Mpeni	#N/A	#N/A	7.63	0.00	-99.43
Pump NsuwaziIntake	#N/A	#N/A	34.43	0.00	-4.70
Pipe 87	600	45	0.00	0.00	0.00
Pipe 133	40	29	0.00	0.00	0.00
Pipe 76	195	102	0.17	0.02	0.01
Pipe 104	387	102	0.20	0.02	0.01
Pipe 89	60	100	0.26	0.03	0.03
Pipe 103	430	102	0.45	0.06	0.05
Pipe 95	176	102	0.46	0.06	0.05
Pipe 126	400	57	0.10	0.04	0.06
Pipe 85	1612	100	0.52	0.07	0.08
Pipe 94	520	102	0.82	0.10	0.15
Pipe 74	205	102	0.85	0.10	0.16
Pipe 137	300	45	0.10	0.07	0.18
Pipe 82	897	102	1.13	0.14	0.27
Pipe 60	718	29	0.04	0.07	0.30
Pipe 65	180	45	0.14	0.09	0.31
Pipe 83	109	100	1.13	0.14	0.39
Pipe 81	1953	100	1.13	0.14	0.39
Pipe 84	109	100	1.13	0.14	0.39
Pipe 106	1462	57	0.34	0.13	0.49

Table 7: Pipes with Excessive Unit Head Loss from EPANET Analysis of Existing WDS

Network Table - Links

Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km
Pipe 105	1296	45	1.03	0.65	12.13
Pipe 49	120	50	1.01	0.51	13.01
Pipe 107	398	29	0.34	0.52	13.24
Pipe 131	150	29	0.36	0.54	14.33
Pipe 45	100	45	1.18	0.74	15.54
Pipe 44	756	29	0.37	0.57	15.70
Pipe 18	2303	148	28.32	1.65	16.91
Pipe 19	1294	148	28.32	1.65	16.91
Pipe 17	460	148	28.32	1.65	16.91
Pipe 55	103	29	0.40	0.60	17.46
Pipe 69	120	75	4.74	1.07	22.56
Pipe 68	157	75	4.82	1.09	23.22
Pipe 135	292	29	0.48	0.73	24.93
Pipe 136	40	25	0.24	0.49	26.54
Pipe 67	80	81	5.30	1.03	26.69
Pipe 51	92	50	1.52	0.77	27.68
Pipe 28	298	150	28.32	1.60	29.54
Pipe 61	655	29	0.61	0.92	38.62
Pipe 88	150	29	0.61	0.93	39.10
Pipe 56	70	25	0.32	0.66	45.99
Pipe 71	100	25	0.48	0.98	96.53
Pipe 129	60	21	0.44	1.26	101.36

It can also be observed from Table 6 and Table 7 that there is a direct relationship between pipe diameter and unit head loss. The smaller the pipe diameter the higher the unit head loss. Similarly the bigger the pipe diameter the lower the unit head loss.

High unit head loss above 5m per kilometre is an indicator of undersized pipe diameter with respect to water flow rate (SRWB, 2009). Under normal circumstances such undersized pipe diameters ought to be optimised so as to bring unit head loss below the recommended standard of 5m/km. Literature review (Table 1) indicates that undersized pipes are prone to beam failure that leads to leakage which this study was trying to address (FCMNRC, 2003). In light of the fact that Thyolo Boma water distribution system had very high pressures, high unit head loss in this context, plays a great positive role in reducing excess pressure (Arnalich, 2011). Therefore reducing pipe diameters in areas with very high pressures, to deliberately introduce high unit head loss, could be part of pressure management as long as the installed pipes were of adequate nominal pressure to absorb maximum dynamic operating pressures.

4.2.2 Pipe Burst Analysis

SRWB provides for 0.55bursts/km/year as one key performance indicator (SRWB, 2015). A review of the Faults Register for Thyolo from 2011 to 2014 gave the following results shown in Figure 9 below.

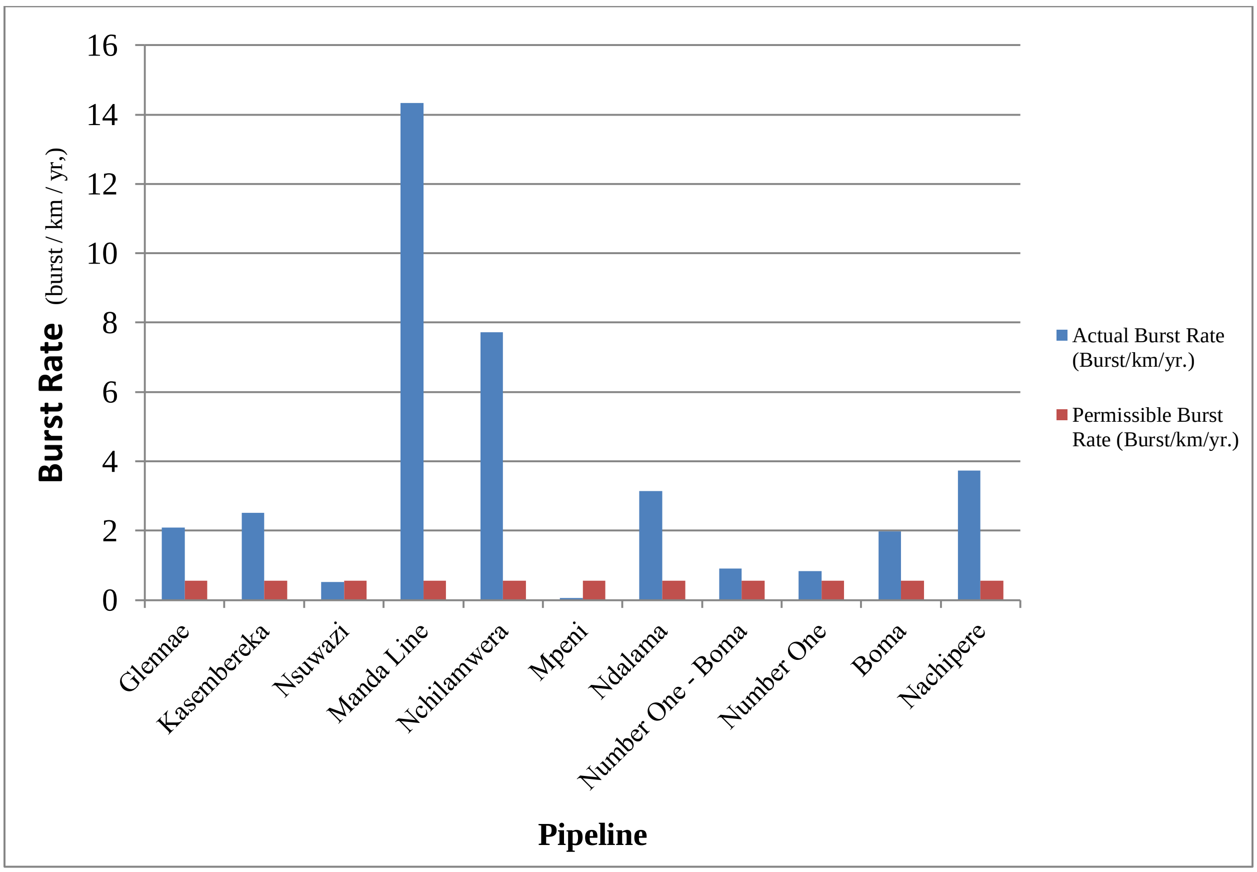

Figure 9: Pipe Burst Results for Selected Major Pipelines

From Figure 8 above, the following could be observed:

- Only two pipelines, Nsuwazi – Number One and Mpeni – Number One, which also happen to be transmission lines from the two Water Treatment Plants at Mpeni and Nsuwazi, met the SRWB performance criteria of 0.55bursts/km/year with 0.55bursts/km/year.
- Two pipelines, Number One pipelines and Number One – Boma were slightly above the permissible pipe burst rate.
- Seven pipelines, representing more than 50 percent of the total pipeline in the water distribution network, registered excessively high pipe burst rates.

4.2.2.1 *Boma – Glennae Pipeline*

Pipe burst analysis in Figure 8 above shows failure rate of 2.08burts/km/year for this pipeline. It was observed from pipe reticulation records and the topographic survey, that different sections on this line were fitted with the lowest to medium pressure rated PVC (Polyvinyl Chloride) pipes. These are class 6 PVC pipes that have a nominal pressure rating of 6 bars or 60m head and a maximum allowable operating pressure of 48m (4.8 bars) while the rest was fitted with class 10 PVC pipes with 10bars or 100m nominal pressure and 80m maximum allowable operating pressure (refer Table 3). EPANET simulation results for this pipeline registered a pressure range of 77m to 99m for the section with a mixture of class 6 and class 10 PVC pipes (Nodes 118 to 123) while the rest of the pipeline towards the end (Nodes 123 to 138) had very high pressures ranging from 99m to 141m for the same nominal and maximum allowable pressures highlighted above. It was established that of the 76 pipe failures recorded on the entire pipeline, 59% (45 failures) occurred on the upstream of node 123 while 41% (31 failures) downstream. 93% (42 failures) of pipe bursts upstream of node 123 with mixed pipe classes were concentrated on class 6 PVC pipes compared to 7% (3 failures) on class 10 PVC pipes because recorded operating pressures were way above its limits/pressure capacity while those on class 10 pipes were either within or slightly above allowable limit but within nominal pressure limit. It was reported that all the 110mm PVC pipes including class 6 pipes were installed on temporary basis while relocating the pipeline during the reconstruction of the Limbe-Thyolo-Muloza road in the mid-1990s in an effort to quickly restore supply to Glennae area while the right pipes were being procured but ended up being forgotten. It was also established that all the 31% pipe failures downstream of node 123 were solely as a result of operating pressure being above both nominal and maximum allowable operating pressures. It was reported during the topographic survey that this high number of pipe failures have been registered just recently from year 2011 when Glennae steel elevated tank earmarked for Thyolo secondary school and surrounding settlements was bypassed due to wear and tear beyond repair hence no break-pressure facility initially assumed by this tank.

4.2.2.2 *Number One – Kasembereka Pipeline*

This line comprised Asbestos cement (AC) class10 pipes, PVC pipes class 10, and Ductile Cast Iron class G pipes. From Figure 5, the pipeline had pressure range of 19.7m upstream on high elevations (nodes 82 to 87) to 160.2m downstream in low elevations (nodes 88 to 91) respectively. It was observed that high pipe bursts registered along this pipeline was generally concentrated on a section fitted with AC pipes. Burst pipe analysis as given in Figure 8 for this pipeline is 2.52bursts/km/year and the age (from fixed asset records and reticulation maps) for the pipeline is 46 years, 100mm diameter, 100m nominal pressure, 80m maximum allowable operating pressure and 2.07km long. The section covered by 110mm diameter PVC pipes (898m long) was initially part of this AC pipeline but has been and continues to be gradually replaced by PVC pipe material. Mordak and Wheeler (1988) established that pressure resistance of AC pipes deteriorates linearly to a factor of 10.5 for pipes aged above 40 years. This could be true of all AC pipes in all SRWB water distribution systems that are being replaced and phased out including this one hence the high concentration of pipe failures. The lower part of the pipeline that also corresponded to the high pressure section fitted with ductile cast iron class G pipes, with 213m nominal pressure and 170m maximum allowable operating pressure against recorded pressure heads varying from 102m to 160.2m. No burst had been recorded under this section since installation other than leaking sluice and air valves implying that operating pressures are within allowable limits.

4.2.2.3 *Nsuwazi – Number One Pipeline*

This was a 5km pumping line comprising 150mm diameter PN16 galvanised iron pipes, 150mm diameter class G cast iron pipes and 150mm diameter class10 Asbestos Cement pipes. All three pipe types had been installed for 46 years. The pipeline registered 13 pipe failures during the period under review which translated to 0.52bursts/km/year. All these failures occurred on AC pipe section between nodes 23 and 24 with operating pressure heads of 58.5m and 9.5m for nodes 23 and 24 respectively. The highest pressure along the entire pumping line and the whole WDS was 164.1m at node 21 which fell within the ductile cast iron pipe section. Most probably the high pressure pipe was designed for this section to contain the high pressures and avoid failure as similarly done on Number One-Kasembereka line. It was observed that despite increasing pipe failures along the AC pipe section, both EPANET simulated operating pressures and measured pressures were well within recommended limits of 100m and 80m for nominal and maximum allowable operating pressures respectively. The pipe failures along this pumping main have been increasing during the last 12 years while it was rare prior to this period (SRWB Mulanje, 2014a). This could be linked to exponential deterioration of AC pipe material over time as earlier alluded to.

4.2.2.4 *Boma – Nchilamwera Pipeline*

The 3.7km long pipeline comprised 90mm, 50mm and 32mm diameter class10 PVC pipes. The oldest part of the pipeline was the 50mm diameter section with 35 years since installation as at end of year 2015. The others were 2 years and 5 years for 90mm and 32mm diameter pipes respectively. The line recorded 143 pipe failures during the period under review translating to failure rate of 7.73bursts/km/year compared to standards of 0.55bursts/km/year for SRWB.

EPANET analysis gave a pressure range of 49m to 118m on the upstream and downstream of the pipeline (i.e. nodes 47, 55, 61, 66 through 68 on EPANET network map). Nominal and maximum allowable operating pressures for these pipes were 100m and 80m respectively. This meant that parts of the pipeline (from node 61 through 68) fell above the allowable operating pressure limit leading to pipe failures. It was observed during topographic survey that there is an elevated pressed steel known as DC's tank at node 47 from which this pipeline was fed but now bypassed because the tank is obsolete due to worn out panels as a result of corrosion. The tank, serving as a service reservoir for four locations including Nchilamwera, also served as a Break-Pressure tank (BPT) bringing inflow pressure from Number One main tank to atmospheric pressure (zero pressure) hence reducing all subsequent downstream pressures in pipelines fed from it. It could therefore be construed that the pipeline under review was properly designed with adequate pressures within required limits. The main factor causing high pressures in Nchilamwera pipeline was isolation of pressure-break facility that was initially assumed by the bypassed DC's tank hence high pipe breakage rate.

4.2.2.5 *Manda Line Pipeline*

This pipeline runs from nodes 113, 114 through 133. It comprised 50mm and 32mm diameter class10 PVC pipes laid 6 years earlier as at year 2015. During the period under review, the line recorded 49 pipe failures on as short as 0.7km stretch translating to 14.33bursts/km/year failure rate. This is the highest failure rate within the water distribution system. Pressure range for this pipeline from EPANET analysis was 77m to 80m whereas nominal pressure and maximum allowable operating pressure for class 10 PVC pipes were 100m and 80m. This means pressures obtained are within limits. In this regard, excess operating pressure and loss of hydraulic capacity due to pipe age (years of service) can be ruled out as cause of pipe failure.

During the topographic survey, it was observed that the pipes were exposed to the surface or thinly buried in the ground. Some sections of the pipeline were running exposed in gullies while loosely buried by either thin layers of soil or rocks. The explanation for the gully trenches was that backfill material had been eroded by storm water that followed weaker ground along the trench as pipeline runs along a steep slope while it was reported that the loose and thinly buried sections was as a result of the trench passing through ground with mass rock where excavation in rock was impossible. The result is that the pipes are often either vandalised by people or crushed by axial load from passing vehicles and animals such as cattle hence the highest pipe breakage rate. Therefore excess working pressure was ruled out as the main cause of such a high pipe failure rate.

4.2.2.6 *Boma Pipelines*

Boma is the main supply area because nearly 70 percent of settlements are concentrated here hence the largest demand and the widest pipe network. The area has several pipelines most of which are short to medium length. Pipe characteristics including class 10 PVC pipelines with diameters ranging from 32mm to 160mm (as given in reticulation maps and fixed asset records), age of service from 4 to 44 years and lengths from as short as 100m to 2.5km; G.I pipes PN16 with two pipe sizes of diameters 100mm and 50mm, nominal pressure 160m, maximum allowable pressure of 128m, and 46 years of service.

The EPANET simulated pressure for this area ranged from 35mto 87m. Minimum nominal pressure and allowable operating pressure for the least pressure rated pipe are 100m and 80m respectively. Only 2 out of 43 nodes (111 and 136) in the area representing 5% had dynamic operating pressures (86.64m and 84.64m respectively) above the maximum allowable operating pressure of 80m but below nominal pressure. The area recorded a total of 148 pipe failures translating to 1.97bursts/km/year which is high with regard to SRWB standard of 0.55bursts/km/year. It was reported during the survey that the bulk of pipe breakage in this area are caused by two main factors; road equipment (graders) crushing pipes during routine road maintenance; surge pressure (water hammers) arising from rapid opening and closure of valves during maintenance of water mains and while resuming water supply when supply was interrupted for one reason or the other. It was also observed that the area has very old G.I pipes (46 year of service) that have corroded with their bore inside reduced by zinc and other sediments hence weak to withstand pressure surges. Some section of AC pipes that are as old as G.I pipes above yield to pressure surges as a result of deterioration pipe material to resist stresses with time,

4.2.2.7 *Nchilamwera - Nachipere Pipeline*

All pipes are PVC class10 with diameters ranging from 32mm to 50mm and age of service of 5 years and 32 years respectively. Simulated pressure ranged from 74m to 91m compared to maximum allowable operating pressure and nominal pressure of 80m and 100m respectively. The pipeline recorded a total of 26 pipe failures translating to a burst rate of 3.73/km/year for a 1.393km pipeline. This was a high burst rate compared to SRWB standard of 0.55/km/year.

Nachipere line is connected to Nchilamwera main distribution line hence the cause of high pressures for Boma-Nchilamwera pipeline applies. The main cause of pipe failures in this area is high operating pressures.

4.2.2.8 *Number One Pipelines*

This is the second largest water supply area at Thyolo. It has several pipelines with diameters ranging from 50mm to 200mm. Pipe types include PVC class6, PVC class10, G.I and AC pipes. Service ages of the pipes range from 13 to 46 years. The pipeline recorded a total of 21 pipe failures on aggregated total pipeline length of 5km translating a burst rate of 0.83/km/year. This fails short of that of meeting SRWB's standard.

EPANET simulated pressure ranged from 9.4m to 41m which are within recommended dynamic operating pressures of 10m minimum and 48m maximum allowable operating pressure as well as 60m nominal pressure for the least pressure rated class6 PVC pipe and way below the highest rated PN16 G.I pipe with 128m maximum allowable operating pressure and 160m nominal pressure. High operating pressure can therefore be ruled out as the cause of pipe failures. Pressure surge emanating from sudden opening and closing of valves coupled with long service age for some pipes were observed to be main causes of pipe failures in this area.

4.2.3 Validation of EPANET Pressures

Considering that there can always be some errors with both raw data, which in this context is topographic survey data carried out using a GPS and the software processing it, in this case EPANET, it was considered right and proper to carry out actual measurement of pressures on a few selected pipelines to ascertain the accuracy of pressure results from the water distribution system software, EPANET. Figure 10 below shows results of pressures measured on pipelines using pressure gauges mounted on saddle clips clamped to pressure pipes.

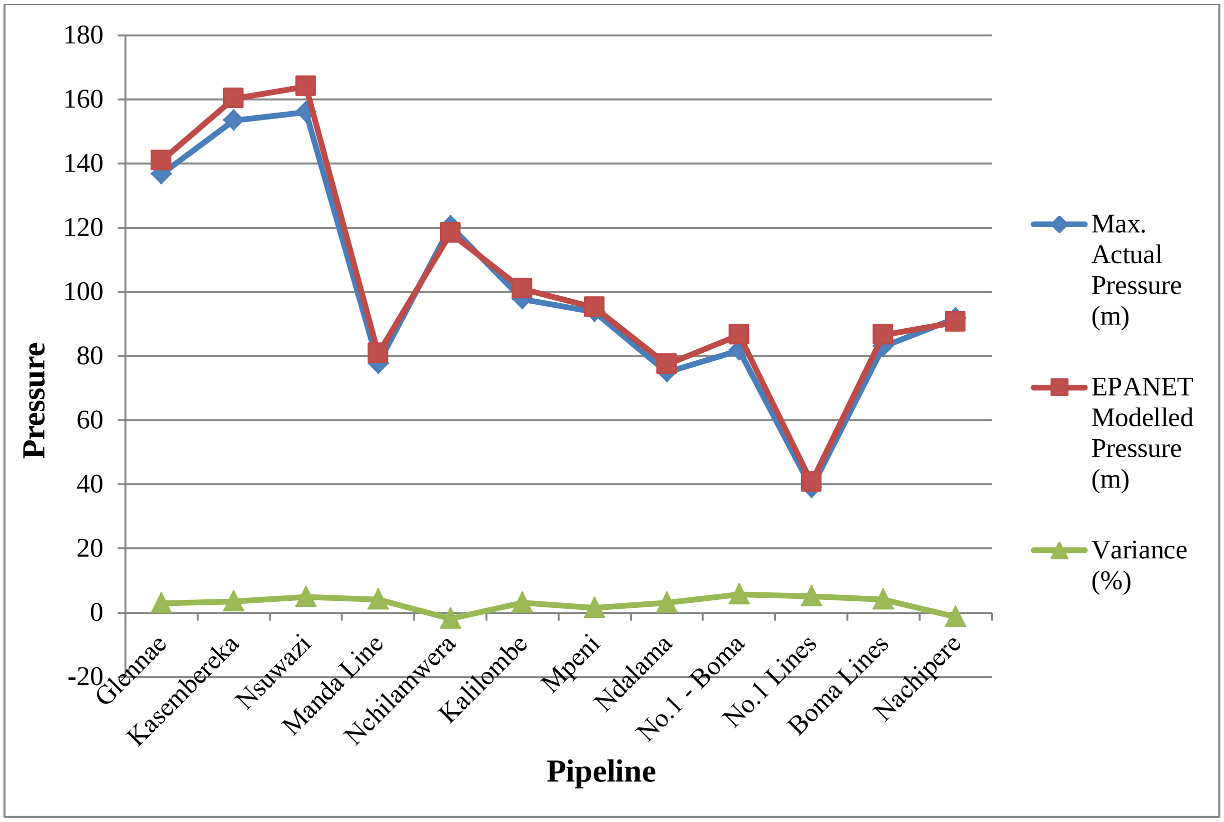

Figure 10: Comparison of EPANET Simulated Pressure and Actual (Measured) Pressure

Minimum and maximum variance between actual and simulated pressure were -1.2m and 5.7m. Similarly variance as percentage ranged from 1.3% to 8.1%. The GPS used has error tolerance of ±5%. This is usually the main source of error for EPANET input data that has significant bearing on analysis results. In Figure 10 above, 11 out of 12 points, representing 92% were within tolerable error based on GPS accuracy whereas 1 out of 12 points, representing 8% inaccuracy, was slightly above range. Average variance was 2.9m representing 3.4%. Therefore overall variance is within tolerable range and close to EPANET simulated pressures. This implies that EPANET simulated the system pressures fairly well and can therefore be relied upon in pressure analyses.

4.2.4 Optimisation of the Water Distribution System

The scope of optimisation of the network for Thyolo WDS can be limited for a number of factors including cost to the utility provider (SRWB) and inconvenience to the water user while making changes to water distribution infrastructure. This study focused on proposals that are more or less of rehabilitation of parts of problematic areas in terms of excessive pressures and high frequency of pipe bursts leading to high water losses. Only one proposal that was a new mechanism in the existing WDS has been made.

The following were proposals made to optimise dynamic (operating) pressures in a few major pipelines that registered excess pressures:

(i) **Reinstatement of Glennae Service Reservoir: Boma-Glennae Pipeline**

This area originally had a pressed steel elevated storage tank mounted on dwarf reinforced concrete walls and it played two roles as Break-Pressure tank (BPT) and treated water reserve. The tank was bypassed in year 2009 upon being declared obsolete because of wear and tear of tank steel panels due to corrosion. As a result, high pressures (Figure 6 above, node 113 to 138,) were introduced to both downstream and upstream areas of the tank since the mechanism to bring high pressures from the main tank at Number One to atmospheric (zero) pressure on entry into the tank was removed. The tank has been reinstated (Tank 80, optimal network). The tank can be repaired by way of replacing some steel panels or completely replacing the whole unit of panels and supports inside it at fairly reasonable cost. Figure 11 shows new results on the optimised pressure map i.e. after the reinstatement of Glennae Service Reservoir.

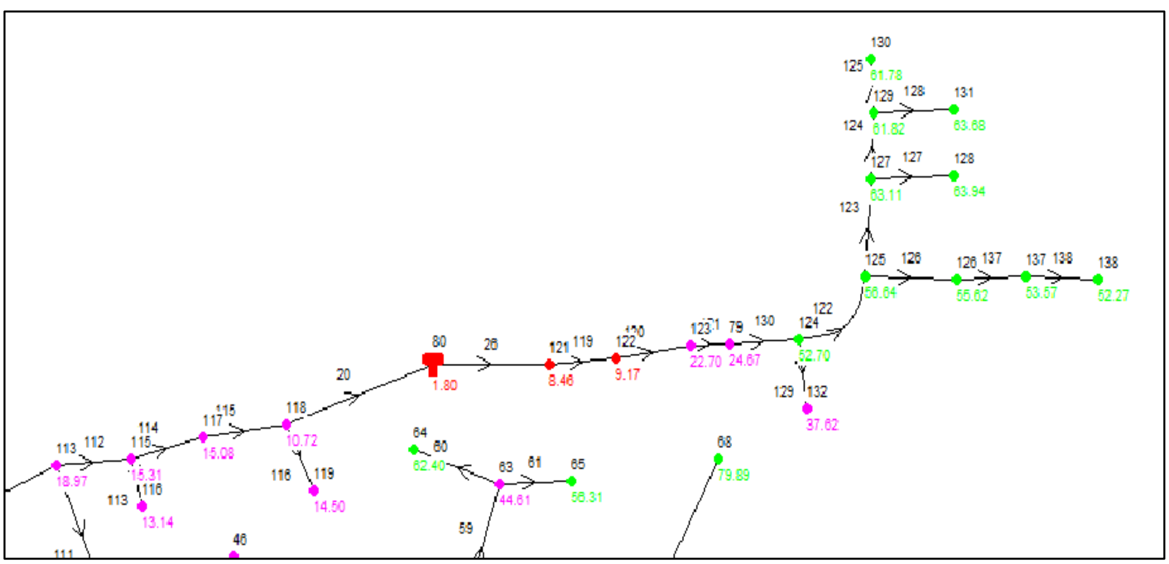

Figure 11: Optimised Pressure Map, Boma-Glennae Pipeline

Figure 11 further depicts a contrast of operating dynamic pressures for the existing and optimised Boma – Glennae pipeline as a representation of Thyolo Boma WDS. It should be noted that the lowest pressure point on the optimised graph (on node 120), coincided with the position where the Glennae tank was reinstated. The nearly zero pressure (1.8m) at this node was due to the fact that the tank could now break the high pressure in the pipeline to atmospheric pressure. It could peak gravity pressure head again as it moves down to low elevation Glennae area.

The reintroduction of Glennae service reservoir (Tank 80, Figure 12) reduced pressures from 70.1m – 140.8m range (existing WDS) to 8.46m – 63.94m (optimised WDS). The section with class 6 PVC pipes (in red and purple, Figure 11) now had maximum dynamic operating pressure of 24.67m and minimum of 8.46m against maximum allowable operating pressure of 48m. This means that, with the exception of pressure surges emanating from rapid valve opening and closure, there would be no or very little influence of dynamic pressure on pipe bursts along this section because simulated dynamic pressures would now be way below the maximum allowable operating pressure. Though the standard minimum pressure along a WDS is 10m as per SRWB Design Criteria, 8.5m is still adequate to deliver desired tap pressure in a non-multi-storey house but not ideal for installation of fire hydrants for firefighting. This pipeline section was not fixed with a fire hydrant hence 8.5m would be adequate.

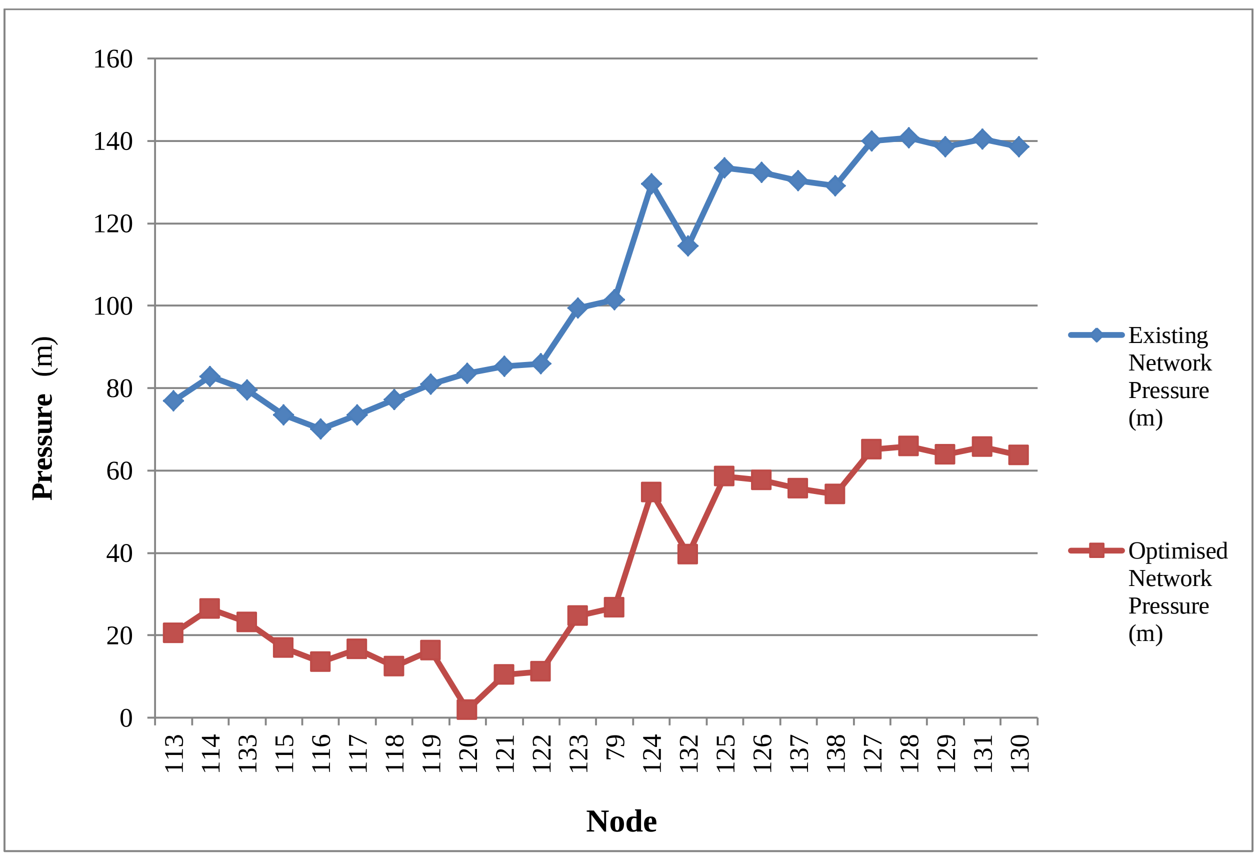

Figure 12: Comparison of Existing and Optimised WDS Pressure

(ii) **Reinstatement of DC's Service Reservoir**

As was the case with Glennae tank, DC's tank (Tank 120, optimised WDS), a pressed steel elevated tank mounted on an 8m brick tower, was bypassed in year 2007 because the steel panels had completely worn out such that the tank could not properly hold water. Therefore it was temporarily isolated from the distribution system to check leakage as well as the risk of collapse and destruction of surrounding public and private property from storm water coming from tank leakage as well as endangering people's lives. Consequently, high pressures (Figure 12, from node 47 to node 68) were introduced to both downstream and upstream areas of the tank since the break-pressure mechanism to bring high pressures from the main tank at Number One to atmospheric (zero) pressure on entry into the tank was removed.

To optimise dynamic pressure the tank was reinstated in the WDS on the EPANET model. The tank could be repaired by way of replacing some steel panels or completely replacing the whole unit of panels and supports inside it at fairly reasonable cost. The following were the optimised results on the pressure map (Tank 120 to Node 68 and 96).

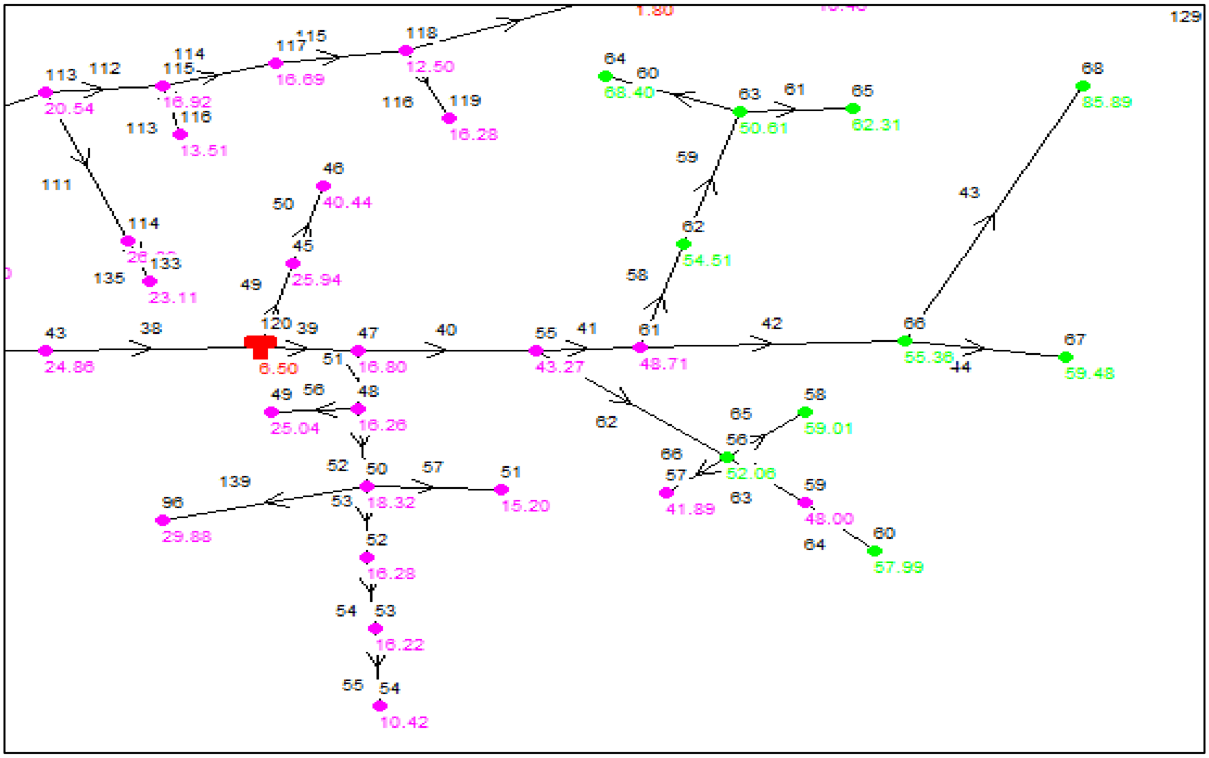

Figure 13: Optimised Pressure Map for Nchilamwera, Part of Boma, Nachipere, Kalilombe and New Hospital Lines

The reintroduction of DC's service reservoir reduced dynamic pressures from 49.3m – 118.4m range (existing WDS) to 10.42m – 85.89m (Figure 13 and Figure 14). All but one node (node 68) had pressures within minimum and maximum allowable operating pressure of 10m and 80m for class 10 PVC pipes. However, the high pressure at node 68 of 85.89m could still be maintained since it was way below the pipe's nominal pressure of 100m of which pipe failures could be contained with proper operation of flow control valves that minimise pressure surges. Alternatively, pipe 43 leading to node 68 could be reduced to 29mm to bring the pressure at node 68 down to 59.89m which would now fall within the allowable operating pressure of 80m for this type of pipe. Nevertheless, water demand around node 68 was still growing hence reducing pipe size would limit flow later on and hence access to potable water for new consumers limited as well.

The tank catered for five pipelines namely; Nchilamwera-Kalilombe (node 61 through node 65), Boma-Nchilamwera (node 47 through node 68), Nchilamwera-Nachipere (node 55 through node 60), Part of Boma (Senior Quarters) from node 47 through node 96), and New Hospital Staff lines (Tank 120 through node 46). Any adjustment to tank 120 that had a bearing on water levels had effect on these pipelines including some nodes upstream of the tank. Below (Figure 13) is a graphic contrast of the existing and optimised distribution network for this section.

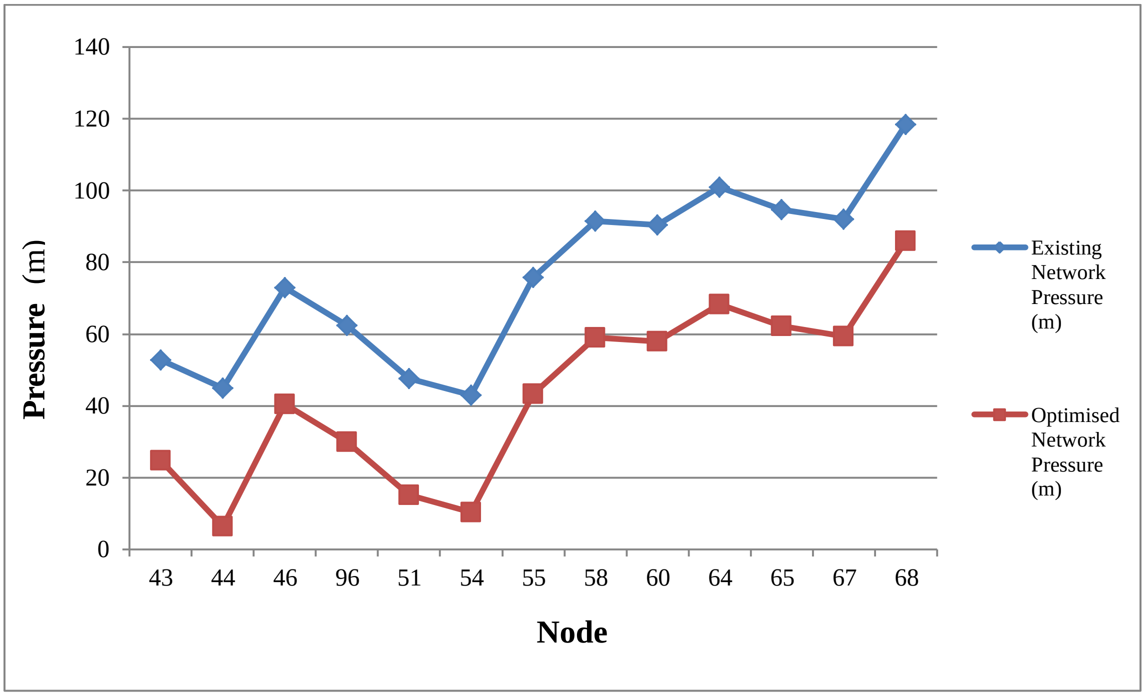

Figure 14: Comparison of Existing and Optimised WDS upon Reinstatement of DC's Tank

(iii) **Installation of Pressure Reducing Valves on Number One – Kasembereka Pipeline**

This was a pipeline with the second highest simulated and measured pressure of 160.2m and 154.4m respectively in the entire WDS basically as a result of being located at the lowest elevations. It had no intermediate service reservoir that would play the role of a Break-Pressure-Tank (BPT) as was the case with Boma-Glennae and Boma-Nchilamwera pipelines. Water demand along this pipeline was very low and declining because some Tea Estates abandoned the utility's service in preference to their own which they developed. Besides this there were no new noticeable settlements along the pipeline that would grow water demand. In this regard it would not be cost effective to construct a service reservoir to double act as BPT. Therefore, two pressure reducing valves (Valves 83 and 85) were now introduced along the pipeline as alternative pressure regulating facilities. Figure 15 is the optimised pressure map from EPANET analysis of Number One-Kasembereka pipeline.

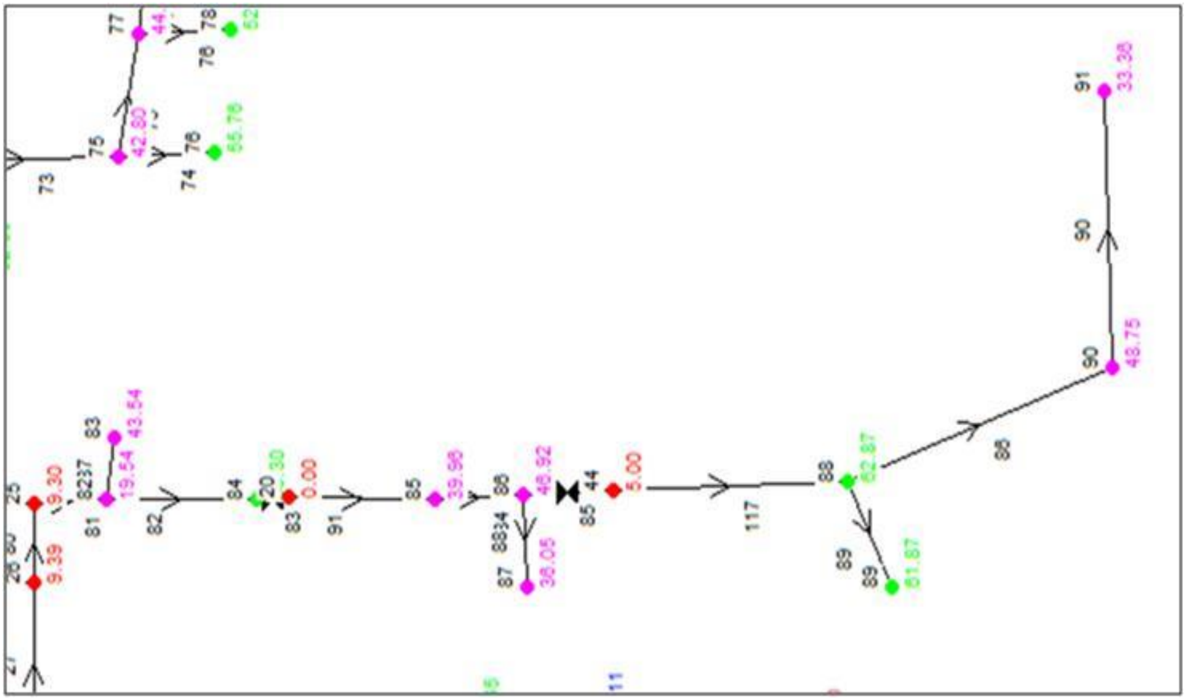

Figure 15: Optimised Pressure Map for Number One - Kasembereka Pipeline

A contrast of pressure change between existing and optimised network results for the pipeline on nodes 84 through 91 now looks (refer to Figure 16) as follows;

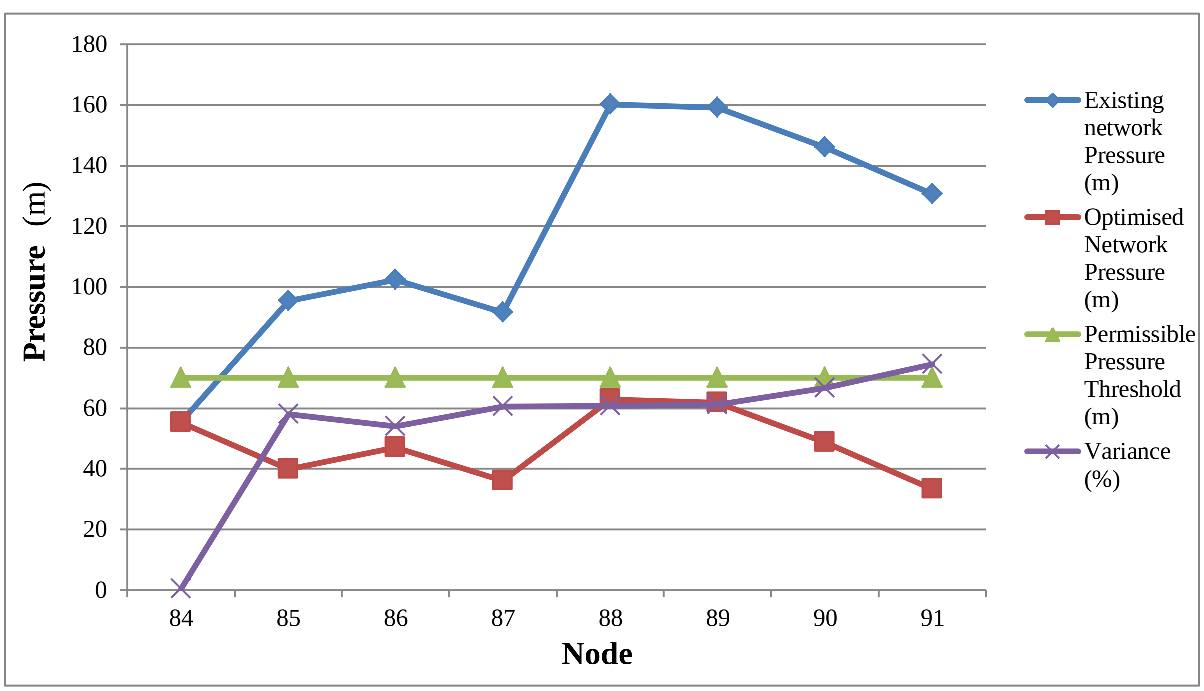

Figure 16: Comparison of Existing and Optimised Pressures for Kasembereka Pipeline

It could be observed from Figure 15 and Figure 16 that it was possible to reduce high pressure in a WDS to allowable levels for sustainability of the system's operational efficiency, which in this case was minimisation of pipe failures through pressure management hence reduction in physical water losses. The PRVs were able to reduce pressure on the pipeline under review by as much as 74.5% to a level that could not burst the existing pipes without making significant changes to the WDS or specific pipeline like major pipe upgrades (upgrading to new pipe of better DN and PN). Also considering that water demand downstream was very low, there was no need to introduce a BPT that would also store water as buffer in case of water supply interruption.

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

5.1 Introduction

This chapter concludes on key findings of the study and makes recommendations on how to improve on some observations made.

5.2 Excess Pressure

This study has shown that Thyolo Boma is experiencing excessive water distribution pressures and this is due to wide variations in topography leading to high pipe breakage rates. The study established that about 42 percent of Thyolo Boma's total water distribution pipeline suffers pipe burst rates of above SRWB's maximum permissible rate of 0.55burst/km/year. Lack of national standard pipe burst rate that can be applicable to all water utilities in Malawi is a challenge to fair analysis of pipe burst rates acceptable to all. This is a gap that requires further study to establish a common (national) standard. Reinstatement of service reservoirs that also play the role of break-pressure facilities and introduction of pressure regulating devices on key pipelines helped to normalise critical pressure to permissible levels.

5.3 Ageing Pipes

Some very old pipes, particularly Asbestos Cement (AC) pipes, with at least 40 years of service, have deteriorated and lost their strength and hence reduced capacity to withstand stresses from the same dynamic pressures they used to contain in early years of service. This is more prominent in sections fitted with AC pipes along Number One-Kasembereka, Number One-Boma and Boma pipelines that continue to register high pipe burst rates. These pipes require total replacement. They have lived their useful design span. Further investigations should be carried out on factors leading fast deterioration of these pipes.

5.4 Poor Pipe Installation Practices

Standard pipe installation practices particularly minimum pipe trench depths are not strictly being observed. This practice, as was observed on Manda Line, is exposing pipes to risk of breakage through vandalism and imposed loads leading to high leakage. Strict observation of construction standards can save the situation

5.5 Velocities and Unit Head Loss

Fifty five percent of Thyolo water distribution system experiences velocities lower than recommended. This is putting the bigger part of the system at risk of corrosion that triggers pipe failure due to reduced strength. Very high unit head losses, though an indicator of pipe under sizing, is in the current state of Thyolo WDS assisting in reducing pressures are also too high as a result of small diameter The utility's planners should always ensure optimal pipe network design to reduce the risk.

5.6 Pipe Sizes

All the 29% of links with higher unit head losses than recommended are an indication that the diameters are small. This is a source of beam failure for pipes and leakage. The utility should include part replacement plans in their future expansion programs.

5.7 Nominal Pressure

Nominal pressure is a factory set or design pressure rating and classification of a pipe. The study has established that pipe bursts were very high in areas where dynamic operating pressures were higher than nominal pressure rating for the pipe. It is therefore recommended that SRWB should consider replacing class 6 PVC pipes with class 10 (10bars or 100m pressure rating) PVC pipes where dynamic pressures are below 80m, replace class 6 and class 10 PVC pipes with class 12 (12bars or 120m pressure rating) PVC pipes, and replace class 6, class 10 and class 12 PVC pipes with class16 (16 bars or 160m pressure rating) PVC pipes if rehabilitation of Glennae tank can prove more costly than replacing pipes.

5.8 Service Reservoirs

Abandoning the two service reservoirs at Glennae and the Boma (DC's tank) by bypassing them raised operating pressures in downstream pipeline which has triggered pipe bursts. This is so because the affected areas are now being supplied direct from the main service reservoir at Number One which unfortunately is at a much higher elevation. Reinstating the two tanks will bring operating pressures to permissible levels.

5.9 Water Distribution System Analysis

EPANET rightly and effectively analysed and mapped hydraulic regimes for the entire existing Thyolo WDS which helped to establish problem areas requiring improvement. The optimised WDS, which now has optimal hydraulic regime levels will ultimately ensure improved operational efficiency hence reduced pipe failures and consequently reduced physical water losses. It is therefore recommended that SRWB should adopt EPANET as a WDS planning and monitoring tool not only at new project planning phase but also for periodic performance review of existing systems to guide in improvement projects decision making.

Two prominent shortfalls in effective implementation of EPANET analysis were observed. These are accuracy of survey equipment and fluctuating water levels on particular days and time whose impact could be seen on variations between actual and EPANET simulated pressures. Therefore simple, affordable and portable survey equipment like GPS of very high precision is an area for further research for technologists. In the case of tanks levels have influence on dynamic pressures; timing of actual pressure measurement must strictly coincide with low an peak demand hours so as to be consistent in taking the highest and lowest operating pressures respectively that can correlate with simulated pressures.

5.10 Recommendation for Further Research

During the study, it was discovered that there are also some operational practices that are contributing to high physical water losses that require further and urgent attention if the water distribution system is to function efficiently in terms of pipe failures. The following areas therefore require further research to quantify their contribution to the physical water losses;

5.10.1 Workmanship

Poor pipeline construction practices like shallow pipeline trenches that result in exposed pipes, pipes being crushed by vehicles, temptation for illegal connections by the general public including some customers as well as vandalism of pipes.

5.10.2 Transient Pressures

Pressure surges in the water distribution system as a result frequent and at times sudden valve opening and closures on account of water interruptions synonymous with Thyolo WDS was found to be one of the causes of pipe failures contributing to increase in physical water losses due to transient pressures or pressure surges in pipes. Further research is therefore required to quantify its magnitude and extent of damage it contributes to pipe failure and hence physical water losses.

5.10.3 Quantification of Water Loss from Burst Pipes

There should be a study to assess and quantify the contribution of water loss from pipe bursts to the total water loss (Non-Revenue Water) of Thyolo Boma WDS for purposes of costing and prioritisation of leakage management programs.

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APPENDICES

Appendix 1: Node Results of Existing WDS from EPANET Analysis

Node ID	Elevation (m)	Base Demand (l/s)	Demand (l/s)	Head (m)	Pressure (m)
Reservoir 2	841	N/A	-34.43	841.00	0.00
Reservoir	998	N/A	-7.63	998.00	0.00
Tank 29	1005	N/A	6.52	1005.50	0.50
Tank 27	1095	N/A	-20.69	1095.60	0.60
Tank. 28	845	N/A	28.36	845.70	0.70
Junc. 25	996	0	0.00	1005.42	9.42
Junc. 26	996	0	0.00	1005.46	9.46
Junc. 24	996	0	0.00	1005.52	9.52
Junc. 30	993	0	0.00	1005.39	12.39
Junc. 8	1083	0	0.00	1095.69	12.69
Junc. 98	990	0.279	0.28	1004.92	14.92
Junc. 82	985	0	0.00	1004.66	19.66
Junc. 101	982	0.356	0.36	1002.78	20.78
Junc. 102	983	0	0.00	1004.39	21.39
Junc. 31	983	0	0.00	1005.02	22.02
Junc. 21	1014	0	0.00	1036.19	22.19
Junc. 32	981	0	0.00	1004.87	23.87
Junc. 93	981	0	0.00	1005.11	24.11
Junc. 11	1069	0	0.00	1094.02	25.02
Junc. 18	1057	0	0.00	1082.92	25.92
Junc. 97	979	0	0.00	1004.93	25.93
Junc. 10	1068	0	0.00	1094.15	26.15
Junc. 94	976	0.210	0.21	1004.81	28.81
Junc. 92	975	0.24	0.24	1004.33	29.33
Junc. 95	973	0	0.00	1005.01	32.01
Junc. 104	971	0	0.00	1003.01	32.01
Junc. 13	1061	0	0.00	1093.38	32.38
Junc. 12	1061	0	0.00	1093.38	32.51
Junc. 107	970	0	0.00	1003.01	33.01
Junc. 105	969	0	0.00	1002.88	33.88

Junc. 108	968	0	0.00	1002.83	34.83
Junc. 99	979	0	0.00	1014.30	35.30
Junc. 106	965	0.186	0.91	1004.80	39.80
Junc. 33	962	0	0.00	1002.88	40.88
Junc. 54	942	0.396	0.40	984.91	42.91
Junc. 83	961	0	0.00	100.66	43.66
Junc. 14	1049	0	0.00	1093.02	44.02
Junc. 15	1048	0	0.00	1092.54	44.54
Junc. 44	946	0	0.00	990.99	44.99
Junc. 51	938	0.481	0.48	985.69	47.69
Junc. 53	938	0	0.00	986.71	48.71
Junc. 48	939	0	0.00	987.75	48.75
Junc. 52	938	0	0.00	986.77	48.77
Junc. 17	1041	0	0.00	1089.82	48.82
Junc. 47	941	0	0.00	990.82	49.29
Junc. 72	937	0.042	0.04	986.90	49.90
Junc. 16	1040	0	0.0..	1090.07	50.07
Junc. 41	941	0	0.00	991.79	50.79
Junc. 50	936	0	0.00	986.81	50.81
Junc. 70	932	0.482	0.48	983.60	51.60
Junc. 74	935	0	0.00	986.62	51.62
Junc. 69	941	0	0.00	993.25	52.25
Junc. 9	1043	0	0.00	1035.56	52.56
Junc. 73	934	0.153	0.15	986.70	52.70
Junc. 43	939	0	0.00	991.77	52.77
Junc. 81	932	1.778	1.78	985.12	53.12
Junc. 6	1042	0	0.00	1096.20	54.20
Junc. 37	937	0	0.00	991.93	54.93
Junc. 84	949	0	0.00	1004.42	55.42
Junc. 35	939	0	0.00	995.03	56.03
Junc. 7	1040	0	0.00	1096.14	56.14
Junc. 75	930	0	0.00	986.35	56.35
Junc. 34	939	0	0.00	995.39	56.36
Junc. 71	933	0.074	0.07	989.61	56.61
Junc. 110	946	0	0.00	1002.81	56.81
Junc. 42	934	0.292	0.29	991.49	57.49
Junc. 49	927	0.323	0.32	984.53	57.53

Junc. 77	928	0	0.00	986.34	58.34
Junc. 45	931	0	0.00	989.43	58.43
Junc. 23	950	0	0.00	1008.45	58.45
Junc. 109	944	0	0.00	1002.81	58.81
Junc. 100	927	0	0.00	986.81	59.18
Junc. 38	931	0.659	0.66	990.38	59.38
Junc. 19	1014	0	0.00	1075.14	61.14
Junc. 39	929	0.253	0.25	990.24	61.24
Junc. 36	933	6.276	6.28	994.78	61.78
Junc. 96	924	0.320	0.32	986.37	62.37
Junc. 40	925	0.269	0.27	988.58	63.58
Junc. 134	920	1.416	1.42	985.62	65.62
Junc. 78	920	0.170	0.17	986.34	66.34
Junc. 112	935	0.204	0.20	1002.44	67.44
Junc. 76	917	0.845	0.85	986.32	69.32
Junc. 116	920	0.195	0.19	990.09	70.09
Junc. 46	913	1.011	1.101	985	72.93
Junc. 117	918	0	0.00	991.47	73.47
Junc. 115	918	0	0.00	991.50	73.50
Junc. 57	911	0.297	0.30	985.38	74.38
Junc. 55	913	0	0.00	988.76	75.76
Junc. 113	916	0	0.00	992.95	76.95
Junc. 118	913	0	0.00	990.18	77.18
Junc. 103	911	1.033	1.03	988.67	77.67
Junc. 135	906	0	0.00	984.52	79.52
Junc. 59	902	0	0.00	982.50	80.50
Junc. 119	909	0.168	0.17	989.97	80.97
Junc. 61	907	0	0.00	988.20	81.20
Junc. 114	909	0	0.00	991.80	82.80
Junc. 63	903	0	0.00	986.10	83.10
Junc. 120	905	0	0.00	988.62	83.62
Junc. 56	901	0	0.00	985.55	84.55
Junc. 136	895	0.341	0.34	979.63	84.63
Junc. 121	903	0	0.00	988.27	85.27
Junc. 122	902	0	0.00	987.98	85.98
Junc. 111	916	0.247	0.25	1002.64	86.64
Junc. 62	900	0.319	0.32	987.00	87.00

Junc. 66	900	0	0.00	987.85	87.85
Junc. 60	889	0.301	0.30	979.48	90.48
Junc. 87	907	0.612	0.61	998.48	91.48
Junc. 58	894	0.142	0.14	985.50	91.50
Junc. 67	884	0.374	0.37	975.97	91.97
Junc. 3	1005	0	0.00	1097.43	92.43
Junc. 5	1004	0	0.00	1097.10	93.10
Junc. 65	866	0.608	0.61	960.80	94.80
Junc. 4	1002	0	0.00	1097.17	95.17
Junc. 85	909	0	0.00	1004.38	95.38
Junc. 123	888	0	0.00	987.51	99.51
Junc. 64	885	0.044	0.04	985.89	100.89
Junc. 79	886	0	0.00	987.49	101.49
Junc. 86	902	0	0.00	1004.34	102.34
Junc. 132	865	0.438	0.44	979.44	114.44
Junc. 68	866	0.503	0.50	984.38	118.38
Junc. 138	856	0.105	0.10	985.52	129.09
Junc. 124	856	0	0.00	985.52	129.52
Junc. 137	855	0	0.00	985.39	130.39
Junc. 91	869	0.252	0.25	999.70	130.70
Junc. 126	852	0	0.00	985.46	133.46
Junc. 130	844	0.185	0.19	982.60	132.60
Junc. 129	844	0	0.00	982.63	138.63
Junc. 127	844	0	0.00	983.93	139.93
Junc. 131	842	0.214	0.21	982.50	140.50
Junc. 128	843	4.037	4.04	985.75	140.75
Junc. 90	854	0	0.00	1000.09	146.09
Junc. 89	845	0.264	0.26	1004.21	159.21
Junc. 88	844	0	0.00	1004.21	160.21
Junc. 22	847	0	0.00	1011.11	164.11

Appendix 2: Link results of Existing WDS from EPANET Analysis

Link ID	Length (m)	Diameter (mm)	Flow (l/s)	Velocity (m/s)	Unit Head Loss (m/km)	Friction Factor	Status
Pump Nsuwazi	N/A	N/A	6.08	0.00	-165.41	0.00	Open
Pump Mpeni	N/A	N/A	7.63	0.00	-99.43	0.00	Open
Pump Nsuwazi (intake)	N/A	N/A	34.43	0.00	-4.70	0.00	Open
Pipe 87	600	45	0.00	0.00	0.00	0.000	Open
Pipe 133	40	29	0.00	0.00	0.00	0.000	Open
Pipe 76	195	102	0.17	0.02	0.01	0.037	Open
Pipe 104	387	102	0.20	0.02	0.02	0.036	Open
Pipe 89	60	100	0.26	0.03	0.03	0.045	Open
Pipe 103	430	102	0.45	0.06	0.05	0.032	Open
Pipe 95	176	102	0.46	0.06	0.05	0.032	Open
Pipe 126	400	57	0.10	0.04	0.06	0.037	Open
Pipe 85	1612	100	0.52	0.07	0.08	0.036	Open
Pipe 94	520	102	0.82	0.10	0.15	0.029	Open
Pipe 74	205	102	0.85	0.10	0.16	0.029	Open
Pipe 137	300	45	0.10	0.07	0.18	0.036	Open
Pipe 82	897	102	1.13	0.14	0.27	0.028	Open
Pipe 60	718	29	0.04	0.07	0.30	0.038	Open
Pipe 65	180	45	0.14	0.09	0.31	0.034	Open
Pipe 83	109	100	1.13	0,14	0.39	0.037	Open
Pipe 81	1953	100	1.13	0.14	0.39	0.037	Open
Pipe 84	109	100	1.13	0.14	0.39	0.037	Open
Pipe 106	1462	57	0.34	0.13	0.49	0.031	Open
Pipe 125	69	45	0.19	0.12	0.50	0.033	Open
Pipe 132	238	45	0.19	0.12	0.51	0.033	Open
Pipe 42	685	81	0.88	0.17	0.51	0.028	Open
Pipe 122	105	148	4.54	0.26	0.57	0.024	Open
Pipe 78	937	102	1.76	0.22	0.60	0.026	Open
Pipe 96	480	45	0.21	0.13	0.63	0.032	Open
Pipe 53	52	57	0.40	0.16	0.65	0.030	Open
Pipe 128	200	45	0.21	0.13	0.66	0.032	Open
Pipe130	2912	148	4.98	0.29	0.68	0.023	Open

Pipe 119	427	148	4.98	0.29	0.68	0.023	Open
Pipe 114	43	148	5.15	0.30	0.72	0.023	Open
Pipe 108	204	45	0.25	0.16	0.86	0.031	Open
Pipe 77	182	100	1.76	0.22	0.88	0.035	Open
Pipe 90	440	45	0.25	0.16	0.89	0.031	Open
Pipe 97	14	148	5.85	0.34	0.91	0.023	Open
Pipe 75	10	100	1.93	0.25	1.05	10.035	Open
Pipe 21	2530	150	6.08	0.34	1.05	0.026	Open
Pipe 134	3	150	-5.61	0.32	1.07	0.031	Open
Pipe 46	124	57	0.52	0.20	1.08	0.029	Open
Pipe 70	168	40	0.15	0.12	1.17	0.062	Open
Pipe 66	144	45	0.30	0.19	1.21	0.031	Open
Pipe 22	2408	150	6.08	0.34	1.22	0.030	Open
Pipe 102	36	150	6.27	0.35	1.29	0.030	Open
Pipe 48	235	50	0.29	0.15	1.30	0.058	Open
Pipe 139	312	45	0.32	0.20	1.38	0.030	Open
Pipe 3	606	148	7.68	0.44	1.49	0.022	Open
Pipe 5	300	148	7.63	0.44	1.49	0.022	Open
Pipe 1	170	148	7.63	0.44	1.49	0.022	Open
Pipe 138	200	29	0.10	0.16	1.49	0.034	Open
Pipe23	10	150	6.08	0.34	1.71	0.043	Open
Pipe 12	186	231	28.32	0.68	1.93	0.019	Open
Pipe 10	264	231	28.32	0.68	1.93	0.019	Open
Pipe 16	3570	231	28.32	0.68	1.93	0.019	Open
Pipe 14	1277	231	28.32	0.68	1.93	0.019	Open
Pipe 8	732	231	28.32	0.68	1.93	0.019	Open
Pipe 93	53	150	7.88	0.45	1.97	0.029	Open
Pipe 109	185	40	0.20	0.16	1.99	0.059	Open
Pipe 41	278	81	1.85	0.36	2.03	0.025	Open
Pipe 73	129	100	2.77	0.35	2.05	0.032	Open
Pipe 54	31	45	0.40	0.25	2.05	0.029	Open
Pipe 92	150	150	8.09	0.46	2.07	0.029	Open
Pipe 124	622	45	0.40	0.25	2.08	0.029	Open
Pipe 9	51	250	28.32	0.58	2.45	0.036	Open
Pipe 15	101	250	28.32	0.58	2.45	0.036	Open
Pipe 13	199	250	28.32	0.58	2.45	0.036	Open
Pipe 11	52	250	28.32	0.58	2.45	0.036	Open

Pipe 7	15	250	28.32	0.58	2.46	0.036	Open
Pipe 30	145	185	18.41	0.68	2.57	0.020	Open
Pipe 2	28	150	7.63	0.43	2.60	0.041	Open
Pipe 6	34	150	7.63	0.43	2.60	0.041	Open
Pipe 4	24	150	7.63	0.43	2.60	0.041	Open
Pipe 111	392	45	0.48	0.30	2.93	0.028	Open
Pipe 86	1400	40	0.25	0.26	2.94	0.057	Open
Pipe 43	1084	45	0.50	0.32	3.20	0.028	Open
Pipe 57	341	50	0.48	0.24	3.29	0.054	Open
Pipe 29	20	200	18.65	0.59	3.36	0.037	Open
Pipe 58	351	57	0.97	0.38	3.42	0.026	Open
Pipe 116	60	29	0.17	0.25	3.57	0.031	Open
Pipe 79	409	75	1.78	0.40	3.66	0.033	Open
Pipe 80	10	150	9.22	0.52	3.70	0.040	Open
Pipe 98	582	148	12.56	0.73	3.75	0.020	Open
Pipe 40	404	81	2.59	0.50	3.79	0.024	Open
Pipe 33	90	148	12.87	0.75	3.93	0.020	Open
Pipe 120	113	102	4.98	0.61	4.15	0.022	Open
Pipe 118	83	102	4.98	0.61	4.15	0.022	Open
Pipe 117	378	102	4.98	0.61	4.15	0.022	Open
Pipe 121	6	102	4.98	0.61	4.15	0.022	Open
Pipe 101	30	150	11.88	0.67	4.22	0.027	Open
Pipe 100	328	150	11.88	0.67	4.22	0.027	Open
Pipe 37	5	102	5.12	0.63	4.36	0.022	Open
Pipe 115	291	102	5.15	0.63	4.41	0.022	Open
Pipe 113	300	29	0.19	0.30	4.70	0.031	Open
Pipe 112	308	102	5.34	0.65	4.72	0.022	Open
Pipe 36	29	102	5.41	0.66	4.84	0.022	Open
Pipe 99	125	150	12.91	0.73	4.92	0.027	Open
Pipe 52	187	57	1.20	0.47	5.04	0.026	Open
Pipe 72	55	100	4.55	0.58	5.14	0.030	Open
Pipe 59	173	45	0.65	0.41	5.17	0.027	Open
Pipe 110	1784	102	5.82	0.71	5.54	0.022	Open
Pipe 34	40	102	6.28	0.77	6.36	0.022	Open
Pipe 38	122	100	5.12	0.65	6.39	0.030	Open
Pipe 62	491	45	0.74	0.47	6.54	0.022	Open
Pipe 31	22	150	12.56	0.71	6.56	0.038	Open

Pipe 35	445	102	6.59	0.81	6.97	0.021	Open
Pipe 27	6	200	27.87	0.89	7.07	0.035	Open
Pipe 32	1007	148	18.17	1.06	7.44	0.019	Open
Pipe 47	195	29	0.27	0.41	8.53	0.029	Open
Pipe 127	20	81	4.04	0.78	8.64	0.022	Open
Pipe 39	78	81	4.11	0.80	8.93	0.022	Open
Pipe 123	149	81	4.44	0.86	10.29	0.022	Open
Pipe 63	291	29	0.30	0.46	10.50	0.029	Open
Pipe 64	287	29	0.30	0.46	10.50	0.029	Open
Pipe 50	300	45	1.01	0.64	11.56	0.025	Open
Pipe 105	1296	45	1.03	0.65	12.13	0.025	Open
Pipe 49	120	50	1.01	0.51	13.01	0.048	Open
Pipe 107	398	29	0.34	0.52	13.24	0.028	Open
Pipe 131	150	29	0.36	0.54	14.33	0.028	Open
Pipe 45	100	45	1.18	0.74	15.54	0.025	Open
Pipe 44	756	29	0.37	0.57	15.70	0.028	Open
Pipe 18	2303	148	28.32	1.65	16.91	0.018	Open
Pipe 19	1294	148	28.32	1.65	16.91	0.018	Open
Pipe 17	460	148	28.32	1.65	16.91	0.018	Open
Pipe 55	103	29	0.40	0.60	17.46	0.028	Open
Pipe 69	120	75	4.74	1.07	22.56	0.029	Open
Pipe 68	157	75	45.82	1.09	23.22	0.029	Open
Pipe 135	292	29	0.48	0.73	24.93	0.027	Open
Pipe 136	40	25	0.24	0.49	26.54	0.054	Open
Pipe 67	80	81	5.30	1.03	26.69	0.040	Open
Pipe 51	92	50	1.52	0.77	27.68	0.045	Open
Pipe 28	298	150	28.32	1.60	29.54	0.034	Open
Pipe 61	655	29	0.61	0.92	38.62	0.026	Open
Pipe 88	150	29	0.61	0.93	39.10	0.026	Open
Pipe 56	70	25	0.32	0.66	45.99	0.052	Open
Pipe 71	100	25	0.48	0.98	96.53	0.049	Open
Pipe 129	60	21	0.44	1.26	101.36	0.026	Open

Appendix 3: Node Results of Optimised WDS from EPANET Analysis

Node ID	Elevation (m)	Base Demand (l/s)	Demand (l/s)	Head (m)	Pressure (m)
Reservoir 2	841	N/A	-34.43	841.00	0.00
Junc. 20	949	0	0.00	949.00	0.00
Reservoir 1	998	N/A	-7.63	998.00	0.00
Tank. 29	1005	N/A	-13.86	1005.50	0.50
Tank. 27	1095	N/A	-20.69	1095.60	0.60
Tank. 28	845	N/A	28.36	845.70	0.70
Tank. 80	905	0	0.00	906.80	1.80
Junc. 121	903	0	0.00	913.46	10.46
Junc. 122	902	0	0.00	913.17	11.17
Junc. 54	942	0.396	0.40	942.42	4.42
Junc. 41	902	0	0.00	907.00	5.00
Junc. 118	913	0	0.00	925.50	12.50
Junc. 116	920	0.195	0.19	933.51	13.51
Tank 120	946	N/A	10.78	952.50	6.50
Junc. 119	909	0.168	0.17	925.28	16.28
Junc. 51	938	0.481	0.48	947.20	9.20
Junc. 117	918	0	0.00	934.69	16.69
Junc. 25	996	0	0.00	1005.29	9.29
Junc. 26	996	0	0.00	1005.38	9.38
Junc. 115	918	0	0.00	934.92	16.92
Junc. 24	996	0	0.00	1005.52	9.52
Junc. 53	938	0	0.00	948.22	10.22
Junc. 48	939	0	0.00	949.29	10.26
Junc. 52	938	0	0.00	948.28	10.28
Junc. 47	941	0	0.00	951.80	10.80
Junc. 30	993	0	0.00	100.19	12.19
Junc. 50	936	0	0.00	948.32	12.32
Junc. 8	1083	0	0.00	1095.69	12.69
Junc. 98	990	0.278	0.28	1004.00	14.00
Junc. 123	888	0	0.00	912.70	24.70
Junc. 41	941	0	0.00	959.04	18.04
Junc. 49	927	0.323	0.32	959.04	18.04
Junc. 102	983	0	0.00	1002.30	19.30

Junc. 82	985	0	0.00	1004.53	19.53
Junc. 79	886	0	0.00	912.67	26.67
Junc. 101	982	0.356	0.36	1001.86	19.86
Junc. 43	939	0	0.00	958.86	19.86
Junc. 45	931	0	0.00	950.94	19.94
Junc. 31	983	0	0.00	1004.12	21.12
Junc. 113	916	0	0.00	936.54	20.54
Junc. 21	1014	0	0.00	1036.19	22.19
Junc. 32	980	0	0.00	1003.68	22.68
Junc. 37	937	0	0.00	960.11	23.11
Junc. 93	981	0	0.00	1004.39	23.39
Junc. 133	905	0.48	0.48	928.11	23.11
Junc. 96	924	0.320	0.32	947.88	23.88
Junc. 42	934	0.292	0.29	958.73	24.73
Junc. 97	979	0	0.00	1004.01	25.01
Junc. 11	1069	0	0.00	1094.02	25.02
Junc. 18	1057	0	0.00	102.92	25..92
Junc. 10	1068	0	0.00	1094.15	26.92
Junc. 104	971	0	0.00	998.00	27.00
Junc. 114	909	0	0.00	935.39	26.39
Junc. 38	931	0.659	0.66	958.55	27.55
Junc. 107	970	0	0.00	998.00	28.00
Junc. 94	976	0.210	0.21	1004.09	28.09
Junc. 105	969	0	0.00	997.61	28.61
Junc. 92	975	0.24	0.24	1004.13	29.13
Junc. 108	969	0	0.00	997.35	29.35
Junc. 39	929	0.253	0.25	958.42	29.42
Junc. 95	973	0	0.00	1004.09	31.09
Junc. 40	925	0.269	0.27	956.76	31.76
Junc. 13	1061	0	0.00	1093.38	32.38
Junc. 12	1061	0	0.00	1093.51	32.38
Junc. 132	865	0.438	0.44	897.62	32.62
Junc. 91	869	0.252	0.25	902.36	33.36
Junc. 72	937	0.042	0.04	971.37	34.37
Junc. 46	913	1.011	1.01	947.44	34.44
Junc. 99	979	0	0.00	1014.30	35.30
Junc. 33	962	0	0.00	997.60	35.60

Junc. 57	911	0.297	0.30	946.89	35.89
Junc. 87	907	0.612	0.61	943.05	36.05
Junc. 70	932	0.482	0.48	968.07	36.07
Junc. 74	935	0	0.00	971.09	36.09
Junc. 69	941	0	0.00	977.73	36.73
Junc. 73	934	0.153	.15	917.18	37.18
Junc. 55	913	0	0.00	950.27	37.27
Junc. 81	932	1.778	1.78	99.59	37.59
Junc. 106	965	0.186	0.19	1003.88	38.88
Junc. 35	939	0	0.00	978.77	39.77
Junc. 85	909	0	0.00	948.96	39.96
Junc. 75	930	0	0.00	970.83	40.83
Junc. 34	939	0	0.00	979.86	40.86
Junc. 71	933	0.074	0.07	974.08	41.08
Junc. 59	902	0	0.00	944.00	42.00
Junc. 61	907	0	0.00	949.71	42.71
Junc. 77	928	0	0.00	970.82	42.82
Junc. 83	961	0	0.00	1004.53	43.53
Junc. 100	927	0	0.00	970.66	43.66
Junc. 14	1049	0	0.00	1093.02	44.02
Junc. 15	1049	0	0.00	1093.02	44.02
Junc. 63	903	0	0.00	947.61	44.61
Junc. 36	933	6.276	6.28	978.52	45.52
Junc. 56	901	0	0.00	947.06	46.06
Junc. 86	902	0	0.00	948.92	46.92
Junc. 138	856	0.105	0.10	910.27	54.27
Junc. 124	856	0	0.00	910.70	54.70
Junc. 62	900	0.319	0.32	948.51	48.51
Junc. 137	855	0	0.00	910.57	55.57
Junc. 90	854	0	0.00	902.75	48.75
Junc. 17	1041	0	0.00	1089.82	48.82
Junc. 66	900	0	0.00	949.36	49.36
Junc. 16	1040	0	0.00	1090.07	50.07
Junc. 134	920	1.416	1,42	970.09	50.09
Junc. 126	853	0	0.00	910.62	57.62
Junc. 78	920	0.170	0.17	970.81	50.81
Junc. 110	946	0	0.00	997.32	51.32

Junc. 125	852	0	0.00	910.64	58.64
Junc. 60	889	0.301	0.30	940.99	51.99
Junc. 9	1043	0	0.00	1095.56	52.56
Junc. 58	894	0.142	0.14	947.01	53.01
Junc. 109	944	0	0.00	997.33	53.33
Junc. 67	884	0.374	0.37	937.48	53.48
Junc. 76	917	0.845	0.85	970.79	53.79
Junc. 6	1042	0	0.00	1096.20	54.20
Junc. 82	949	0	0.00	1004.29	55.29
Junc. 7	1040	0	0.00	1096.14	56.14
Junc. 65	866	0.608	0.61	922.31	56.31
Junc. 130	844	0.185	0.19	907.78	63.78
Junc. 129	844	0	0.00	907.82	63.82
Junc. 127	844	0	0.00	909.11	65.11
Junc. 23	950	0	0.00	1008.45	58.45
Junc. 128	843	4.037	4.04	908.94	65.94
Junc. 19	1014	0	0.00	1075.14	61.14
Junc. 89	845	0.264	0.26	906.87	61.87
Junc. 112	935	0.204	0.20	996.95	61.95
Junc. 64	885	0.044	0.04	947.40	62.40
Junc. 88	844	0	0.00	906.87	62.87
Junc. 135	906	0	0.00	969.37	63.37
Junc. 131	842	0.214	0.21	907.68	65.68
Junc. 136	895	0.341	0.34	964.10	69.10
Junc. 103	911	1.033	1.03	986.59	75.59
Junc. 68	866	0.503	0.50	945.89	79.89
Junc. 111	916	0.247	0.25	997.15	81.15
Junc. 3	1005	0	0.00	1097.43	92.43
Junc. 5	1004	0	0.00	1097.10	93.10
Junc. 4	1002	0	0.00	1097.17	95.17
Junc. 22	847	0	0.00	1011.11	164.11

Appendix 4: Link Results of Optimised WDS from EPANET Results

Link ID	Length (m)	Diameter (mm)	Flow (l/s)	Velocity (m/s)	Unit Head Loss (m/km)	Friction Factor
Pump Nsuwazi	N/A	N/A	6.08	0.00	-165.41	0.000
Pump 24	N/A	N/A	7.63	0.00	-99.43	0.000
Pump 25	N/A	N/A	34.43	0.00	-4.70	0.000
Pipe 133	40	29	0.00	0.00	0.00	0.000
Pipe 87	600	45	0.00	0.00	0.00	0.000
Pipe 76	195	102	0.17	0.02	0.01	0.037
Pipe 104	387	102	0.20	0.02	0.01	0.036
Pipe 89	60	100	0.26	0.03	0.045	0045
Pipe 103	430	102	0.45	0.06	0.05	0.032
Pipe 95	176	102	0.46	0.06	0.05	0.032
Pipe 126	400	57	0.10	0.04	0.06	0.036
Pipe 117	1612	100	0.52	0.07	0.08	0.036
Pipe 94	520	102	0.85	0.10	0.15	0.029
Pipe 74	205	102	0.85	0.10	0.16	0.029
Pipe 137	300	45	0.10	0.07	0.18	0.036
Pipe 82	897	102	1.13	0.14	0.27	0.028
Pipe 60	718	29	0.04	0.07	0.30	0.038
Pipe 65	180	45	0.14	0.09	0.31	0.034
Pipe 81	1953	100	1.13	0.14	0.39	0.037
Pipe 91	109	100	1.13	0.14	0.39	0.037
Pipe 84	109	100	1.13	0.14	0.39	0.037
Pipe 106	1462	57	0.34	0.13	.49	0.031
Pipe 125	69	45	0.19	0.12	0.50	0.033
Pipe 132	238	45	0.19	0.12	0.51	0.033
Pipe 42	685	81	0.8	0.17	0.51	0.028
Pipe 122	105	148	4.54	0.26	0.57	0.024
Pipe 78	937	102	1.76	0.22	0.60	0.026
Pipe 96	480	45	0.21	0.13	0.63	0.032
Pipe 53	52	57	0.40	0.16	0.65	0.030
Pipe 128	200	45	0.21	0.13	0.66	0.032
Pipe 130	2912	148	4.98	0.29	0.68	0.023
Pipe 119	427	148	4.98	0.29	0.68	0.023
Pipe 108	204	45	0.25	0.16	0.86	0.031

Pipe 77	182	100	1.76	0.22	0.88	0.035
Pipe 90	440	45	0.25	0.16	0.89	0.031
Pipe 75	10	100	1.93	0.25	1.05	0.034
Pipe 21	2530	150	6.08	0.34	1.05	0.026
Pipe 46	124	57	0.52	0.20	1.08	0.029
Pipe 70	168	40	0.15	0.12	1.17	0.062
Pipe 134	3	150	-6.04	0.34	1.19	0.030
Pipe 66	144	45	0.30	0.19	1.21	0.031
Pipe 22	2409	150	6.08	0.34	1.22	0.030
Pipe 3	606	148	7.63	0.44	1.49	0.022
Pipe 5	300	148	7.63	0.44	1.49	0.022
Pipe 1	170	148	7.63	0.44	1.49	0.022
Pipe 138	200	29	0.10	0.16	1.49	0.034
Pipe 23	10	150	6.08	0.34	1.71	0.043
Pipe 12	186	231	28.32	0.68	1.93	0.019
Pipe 10	264	231	28.32	0.68	1.93	0.019
Pipe 16	3570	231	28.32	0.68	1.93	0.019
Pipe 14	1277	231	28.32	0.68	1.93	0.019
Pipe 8	732	231	28.32	0.68	1.93	0.019
Pipe 109	185	40	0.20	0.16	1.99	0.059
Pipe 41	278	81	1.85	0.36	2.03	0.025
Pipe 54	31	45	0.40	0.25	2.05	0.029
Pipe 73	129	100	2.77	0.35	2.05	0.032
Pipe 124	622	45	0.40	0.25	2.08	0.029
Pipe 97	14	148	9.67	0.56	2.31	0.021
Pipe 9	51	250	28.32	0.58	2.45	0.036
Pipe 15	101	250	28.32	0.58	2.45	0.036
Pipe 13	199	250	28.32	0.58	2.45	0.036
Pipe 11	52	250	28.32	0.58	2.45	0.036
Pipe 7	15	250	28.32	0.58	2.46	0.036
Pipe 2	28	150	7.63	0.43	2.60	0.041
Pipe 6	34	150	7.63	0.43	2.60	0.041
Pipe 111	392	45	0.48	0.30	2.93	0.028
Pipe 86	1400	40	0.25	0.20	2.94	0.057
Pipe 43	1084	45	0.50	0.32	3.20	0.028
Pipe 57	341	50	0.48	0.24	3.20	0.054
Pipe 58	351	57	0.97	0.38	3.29	0.026

Pipe 116	60	29	0.17	0.25	3.57	0.031
Pipe 79	409	75	1.78	0.40	3.66	0.033
Pipe 40	404	81	2.59	0.50	3.79	0.024
Pipe 121	6	102	4.98	0.61	4.14	0.024
Pipe 26	83	102	4.98	0.61	4.15	0.022
Pipe 120	113	102	4.98	0.61	4.50	0.022
Pipe 113	300	29	0.19	0.30	4.70	0.031
Pipe 52	187	57	1.20	0.47	5.04	0.026
Pipe 114	43	148	14.91	0.87	5.16	0.020
Pipe 72	55	100	4.55	0.58	5.14	0.030
Pipe 59	173	45	0.65	0.41	5.17	0.027
Pipe 93	53	150	14.10	0.80	5.79	0.027
Pipe 92	150	150	14.31	0.81	5.95	0.027
Pipe 34	40	102	6.28	0.77	6.37	0.022
Pipe 62	491	45	0.74	0.47	6.54	0.027
Pipe 102	36	150	15.88	0.90	7.22	0.027
Pipe 30	145	185	32.58	1.21	7.40	0.018
Pipe 47	195	29	0.27	0.41	8.53	0.029
Pipe 127	20	81	4.04	0.78	8.64	0.022
Pipe 39	78	81	4.11	0.80	0.92	0.022
Pipe 29	20	200	32.82	1.04	9.56	0.034
Pipe 80	10	150	15.44	0.87	9.60	0.037
Pipe 123	149	81	4.44	0.86	10.29	0.022
Pipe 64	287	29	0.30	0.46	10.50	0.029
Pipe 63	291	29	0.30	0.46	10.50	9.029
Pipe 98	532	148	22.91	1.33	11.42	0.019
Pipe 50	300	45	1.01	0.64	11.65	0.025
Pipe 33	90	148	23.65	1.37	12.11	0.019
Pipe 105	1296	45	1.03	0.65	12.13	0.025
Pipe 49	120	50	1.01	0.51	13.01	0.048
Pipe 101	30	150	21.92	1.24	13.11	0.025
Pipe 100	328	150	21.92	1.24	13.11	0.025
Pipe 107	398	29	0.34	0.52	13.23	0.028
Pipe 99	125	150	22.95	1.30	14.28	0.025
Pipe 131	150	29	0.36	0.54	14.33	0.028
Pipe 45	100	45	1.18	0.74	15.54	0.025
Pipe 44	756	29	0.37	0.57	15.70	0.028

Pipe18	2303	148	28.32	1.65	16.91	0.018
Pipe 19	1294	148	28.32	1.65	16.91	0.018
Pipe 17	460	148	28.32	1.65	16.91	0.018
Pipe 55	103	29	0.40	0.60	17.46	0.028
Pipe 32	1007	148	28.95	1.68	17.62	0.018
Pipe 27	6	200	48.26	1.54	19.53	0.033
Pipe 31	22	150	22.91	1.30	19.95	0.035
Pipe 69	120	75	4.74	1.07	22.56	0.029
Pipe 68	157	75	4.82	1.09	23.22	0.029
Pipe 135	292	29	0.48	0.73	24.93	0.027
Pipe 136	40	25	0.24	0.49	26.53	0.054
Pipe 67	80	81	5.30	1.03	26.69	0.040
Pipe 51	92	50	1.52	0.77	27.68	0.045
Pipe 28	298	150	28.32	1.60	29.54	0.034
Pipe 20	378	102	14.74	1.80	30.95	0.034
Pipe 115	291	102	14.91	1.82	31.60	0.019
Pipe 112	308	102	15.10	0.88	5.28	0.019
Pipe 110	1784	102	15.43	1,89	33.67	0.019
Pipe 37	5	102	15.90	1.95	35.60	0.019
Pipe 36	29	102	16.19	1.98	36.81	0.019
Pipe 61	655	29	0.61	0.92	38.62	0.026
Pipe 88	150	29	0.61	0.93	39.10	0.026
Pipe 88	150	29	0.61	0.93	39.10	0.026
Valve 85	N/A	100	0.52	0.07	41.92	0.000
Pipe 35	445	102	17.37	2.13	41.94	46.00
Pipe 56	70	25	0.32	0.66	46.00	0.052
Pipe 38	122	100	15.90	2.02	52.15	0.025
Valve 83	N/A	100	1.13	0.14	55.29	0.000
Pipe 71	100	25	0.48	0.98	96.53	0.049
Pipe 129	60	21	0.44	1.26	101.36	0.026

Appendix 5: Pipe Burst Results for selected major pipelines

Pipeline	Burst Count (No)	Length (km)	Burst/km/5yrs (No/km/5yrs)	Burst/km/yr. (No/km/yr.)	Pressure Range (m)
Boma – Glennae	76	7.303	10.41	2.08	70.1 – 140.8
Number One – Kasembereka	81	6.421	12.61	2.52	19.7 – 160.2
Nsuwazi – Number One	13	5.020	2.59	0.52	9.5 – 164.11
Manda Line	49	0.684	71.64	14.33	77.0 – 80.8
Boma – Nchilamwera	143	3.699	38.66	7.73	49.3 – 118.4
Nchilamwera – Kalilombe	56	1.997	28.04	5.61	81.2 – 100.9
Mpeni – Number One	4	17.271	0.23	0.05	12.7 – 95.2
Number One – Ndalama	33	2.095	15.75	3.15	21.4 -77.7
Number One – Boma	7	1.555	4.50	0.90	9.4 – 86.6
Number One Pipelines	21	5.045	4.16	0.83	9.4 - 40.9
Boma Pipelines	148	15.060	9.83	1.97	34.8 – 86.6
Nchilamwera – Nachipere	26	1.393	18.66	3.73	74.4 – 90.5

Appendix 6: Pipe characteristics and allowable pressures

PIPELINE	PIPE TYPE	DIA. (mm)	LENGTH (m)	AGE (Yrs.)	NOMINAL PRESSURE (PN) (Bars)	(m)	PERMISSIBLE PRESSURE (80% PN) (Bars)
Boma – Glennae	PVC class 10	110	2760	16	10	100	80
	PVC class 6	110	196	16	6	60	48
	PVC class 10	160	1132	16	10	100	80
	PVC class 10	160	2274	42	10	100	80
Number One – Kasembereka	Asbestos Cement PN10	100	2072	46	10	100	80
	PVC class10	110	898	1	10	100	80
	Ductile Cast Iron class G	100	1612	46	21	213	170
Nsuwazi – Number One	Galvanised Iron, PN16	150	26	46	16	160	128
	Ductile Cast Iron class G	150	2530	46	21	213	170
	Asbestos Cement, PN10	150	2408	46	10	100	80
Manda Line	PVC class 10	50	362	6	10	100	80
	PVC class 10	32	292	6	10	100	80
Boma - Nchilamwera	PVC class 10	90	760	35	10	100	80
	PVC class 10	32	1441	5	10	100	80
	PVC class 10	50	1278	35	10	100	80
Nchilamwera – Kalilombe	PVC class 10	63	351	2	10	100	80
	PVC class 10	50	173	2	10	100	80
	PVC class 10	32	1373	2	10	100	80
Mpeni – Number One	Galvanised Iron, PN16	250	416	13	16	160	128
	PVC class 10	250	6034	13	10	100	80
	PVC class 12	160	460	13	12	120	96
	PVC class 16	160	8892	13	16	160	128
	Galvanised Iron, PN16	150	298	13	16	160	128
Number One – Ndalama	PVC class 10	50	2087	32	10	100	80
Number One – Boma	Asbestos Cement,PN10	150	732	46	10	100	80
	PVC class 10	110	823	16	10	100	80
	PVC class 10	200	202	13	10	100	80
	PVC class 10	160	1629	13	10	100	80
	PVC class 10	110	509	13	10	100	80
	Galvanised Iron, PN16	150	22	13	16	160	128
	Asbestos Cement, PN10	100	122	46	10	100	80
Number One Pipelines	PVC class 6	110	696	20	6	60	48
	PVC class 10	200	165	13	10	100	80
	PVC class 10	40	238	20	10	100	80

	Asbestos Cement Class10	100	14	46	10	100	80
	PVC class 10	50	1080	13	10	100	80
	Galvanised Iron, PN16	25	40	13	40	160	128
Boma Pipelines	PVC class 10	160	1097	13	10	100	80
	PVC class 10	160	292	16	10	100	80
	PVC class 10	110	509	13	10	100	80
	PVC class 10	110	2522	16	10	100	80
	PVC class 10	110	387	4	10	100	80
	Galvanised Iron, PN16	100	122	46	16	160	128
	Galvanised Iron, PN16	50	235	46	16	160	128
	PVC class 10	50	100	5	10	100	80
	PVC class 10	63	124	44	10	100	80
	PVC class 10	32	195	15	10	100	80
Nchilamwera – Nachipere	PVC class 10	50	1106	32	10	100	80
	PVC class 10	32	287	5	10	100	80

Appendix 7: Measured Pressure in selected points within Thyolo WDS

PIPELINE	MAXIMUM MEASURED PRESSURE		EPANET SIMULATED PRESSURE	VARIANCE	
	kPa	m	M	m	%
Boma – Glennae	1367	136.7	140.8	4.1	2.9
Number One – Kasembereka	1554	154.4	160.2	5.7	3.6
Nsuwazi – Number One	1590	156	164.11	8.1	4.9
Manda Line	776	77.6	80.8	3.2	4.0
Boma –Nchilamwera	1205	120.5	118.4	-2.1	1.8
Nchilamwera – Kalilombe	978	97.8	100.9	3.1	3.1
Mpeni – Number One	938	93.8	95.2	1.4	1.5
Number One – Ndalama	749	74.9	77.3	2.4	3.1
Number One – Boma	817	81.7	86.6	4.9	5.7
Number One Pipelines	388	38.8	40.7	1.9	5.0
Boma Pipelines	829	82.9	86.6	3.7	4.0
Nchilamwera – Nachipere	917	91.7	90.5	-1.2	1.3
Average Variance				**2.9**	**3.4**