2021-07

Performance optimization of unplanned water distribution networks in fast growing towns: a case study of Mwanza city, Tanzania

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NM-AIST

https://doi.org/10.58694/20.500.12479/1284

PERFORMANCE OPTIMIZATION OF UNPLANNED WATER DISTRIBUTION NETWORKS IN FAST GROWING TOWNS: A CASE STUDY OF MWANZA CITY, TANZANIA

Upendo Paul Shushu

A Dissertation Submitted in Partial Fulfilment of the Requirements for the Masters Degree in Hydrology and Water Resources Engineering of the Nelson Mandela African Institution of Science and Technology

Arusha, Tanzania

July, 2021

ABSTRACT

High Non-Revenue Water (NRW) and unreliable water supply services are major challenges in operations of the water networks in most of the fast-growing cities in developing countries. The present study aims at investigating the extent that the existing distribution network contributes to the prevailing high percentage of the NRW; and explore optimization scenarios focusing on water loss reduction and system improvement in the unplanned network. The measured system flow and pressure were used for water balance assessment, calibration and modelling to simulate different scenarios in order to improve system performance. The results showed 52% of the junctions in the system had high pressure above recommended which contributed to 87% of real loss and 83% of pipes had low velocities below the set thresholds. These indicate that uneven distribution of pressures and velocities are driven by improper topology of both pipe sizing and supply directions in the unplanned network. About 50% of NRW was detected in the study area while the entire network had 37%, thus small areas assessment and pressure management are required. The pressure reduction by optimizing installation of pressure reducing valves and change network topology reduced NRW by 46%. In addition, regular nodal hydraulic analysis and flow modifications performed well when integrated with stochastic town growth for system capacity torelance. The study provides ways for sustainably improving the poorly performing water networks in fast-growing towns. It also recommends methods of integrating pressure management, network topology change and resilient to future demand for attaining a better system performance.

DECLARATION

I, Upendo Paul Shushu do hereby declare that the dissertation work presented in this book entitled "Performance Optimization of Unplanned Water Distribution Network in a Fast-growing Towns; A Case Study of Mwanza City, Tanzania" submitted in partial fulfilment of the requirements for the award of a Master degree of 'Hydrology and Water Resources Engineering of the Nelson Mandela African Institution of Science and Technology, Arusha, Tanzania' is a genuine record of my work carried under the guidance of Dr. Hans Charles Komakech and Dr. David Ferras. I have not submitted the matter embodied this report for the award of any other Degree.

Upendo P. Shushu ____________________________ ______________________

Name and signature of candidate **Date**

The above declaration is comfirmed

Dr. Hans C. Komakech ___________________________ ______________________

Name and signature of supervisor (1) **Date**

Dr. David Ferras ______________________________ ______________________

Name and signature of supervisor (2) **Date**

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CERTIFICATION

This dissertation work entitled "Performance Optimization of Unplanned Water Distribution Network in a Fast-growing Towns; A Case Study of Mwanza City, Tanzania" has been approved by the undersigned research supervisors in partial fulfilment of the requirements for the award of a Master degree of 'Hydrology and Water Resources Engineering of the Nelson Mandela African Institution of Science and Technology, Arusha, Tanzania' to Upendo Paul Shushu.

Dr. Hans C. Komakech ___________________________ ______________________

Name and signature of supervisor (1) **Date**

Dr. David Ferras ______________________________ ______________________

Name and signature of supervisor (2) **Date**

ACKNOWLEDGEMENTS

Words are not enough to express my gratitude to Dr. Hans C. Komakech for his treasured guidance, unwearied encouragement, free discussions and enthusiastic interest throughout my research tenure. His willingness to provide support and create functional environments made me move to accomplish my research. I am also thankful to Dr. David Ferras who has been with me throughout my research, providing supervision and materials to ensure this work was successfully. Without their efforts, it would not have been possible to complete this work.

I am indebted to Prof. Mitthal Kal Kansal, the Head, Water Resources Management and Development, Indian Institution of Technology, Roorkee, India, for his technical guidance for the three months I spent in India doing part of this research. It helped to expand my knowledge in data analysis, presentation skills and improving my research paper and dissertation.

I also thank Eng. Anthony Damian Sanga and his technical team of Mwanza Urban Water and Sanitation Authority (MWAUWASA) for their support. I specifically thank him for giving me access to their water infrastructure for installing the experimental equipment and providing the secondary data which were used to apply different methodologies to test how they work in unplanned Water Distribution Network (WDN).

Sincerely, I acknowledge the WISE-futures, a centre of excellence of Nelson Mandela African Institution of Science and Technology, Tanzania for funding my MSc. studies and this research. I am sincerely thankful to the Director of the Centre, Dr. Hans Komakech and his Deputy Director Dr. Abeid Jande for their facilitation in releasing the funds on time to support the undertaking of this study.

My fellow students in the Water, Environment and Sanitation Department were a great source of inspiration to achieve the objectives of this research. They were encouraging and supportive to me through all the ups and downs and I thank them all.

Lastly, my heartfelt thanks go to my husband Samwel and sons Ezra and Elton for their sacrifices throughout my studies. I understand how they were missing my presence when I was away from them. I also thank my parents, sisters and my brother Zakayo for all the support they gave me. This journey would have been impossible to execute without their support.

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LIST OF ABBREVIATIONS AND SYMBOLS

amsl	Above Mean Sea Level
AWWA	American Water Works Association
BABE	Bursts and Background Estimates
BPT	Break Pressure Tank
CARL	Current Annual Real Losses
CI	Cast Iron
DI	Ductile Iron
DMA	District Metering Area
EWURA	Energy and Water Utilities Regulatory Authority
GA	Genetic Algorithm
GIS	Geographical Information System
GS	Galvanised Steel
HDPE	High-density PE
HGL	Hydraulic Grade Line
HWC	Hazen Williams coefficient
IWA	International Water Association
KPIs	Key Performance Indicators
MNF	Minimum Night Flow
MoW	Ministry of Water
MTTF	Mean Time to Failure
MTTR	Mean Time to Repair
MWAUWASA	Mwanza Urban Water Supply and Sanitation Authority
NAO	National Audit Office

NAWAPO	National Water Policy
NBS	National Bureau of Statistics
NRW	Non-revenue Water
PE	Polyethylene pipes
PRV	Pressure Reducing Valve
PRV	Pressure-Reducing Valves
SCADA	Supervisory Control and Data Acquisition
UC	Unbilled Consumption
UfW	Unaccounted for Water
USEPA	United States of America- Environmental Protection Agency
WDN	Water Distribution Network
WSSAs	Water Supply and Sanitation Authorities

CHAPTER ONE

INTRODUCTION

1.1 Background of the problem

Urban sprawl is a universal challenge due to the high migration of population into towns, causing the overall urban population to proliferate. The United Nations habitat report indicates that continents' rates of urbanization soared from 15% in 1960 to 40% in 2010; projected to reach 60% by the year 2050 and to triple in the next 50 years (Torres, 2010). While the global population growth has drastically increased, urban infrastructure experiences limited improvements (Satterthwaite, 2017). Thus, resulting in unplanned expansion of water infrastructure and the emergence of unplanned 'organic' Water Distribution Networks (WDN). In Tanzania, high rates of urbanization and population growth have resulted in an increased water demand, contrary to the speed of infrastructure expansion (National Water Policy [NAWAPO], 2002). It is further reported population growth due to urbanization causes pressure in existing WDNs and reduces the level of performance of the system. Consequently, water utilities are unable to match the pace of towns' growth and may contribute to high Non-Revenue Water (NRW). The reduced performance causes cost disadvantages, including high NRW and low level of services (Chirisa, 2008). According to the International Water Association (IWA), NRW is the difference between system water input volume and billed authorized water consumption. González-gómez *et al.* (2011) defined NRW as the amount of water which is produced and treated from any source but lost without collection of any revenue. During the distribution of water, NRW principally occurs in three components: Unbilled water consumptions, apparent loss and real loss as per International Water Association (IWA) standard water balance. These challenges are projected to continue magnifying due to ever-growing urban population which needs sharing of already insufficient resources (Collier, 2017).

A global water pressure forecasting indicates significant increments in water demand in urban areas (González-gómez *et al.,* 2011). The projections done by the Organisation for Economic Co-operation and Development in 2018 in United Nations World Water Development Report stated that by 2030, 47% of the world population will live in severe water stress. According to González-gómez *et al.* (2011), the global trend forecast indicates water use growth from 4085 km^3 in 2000 to 6275 km^3 in 2050. In Tanzania, water demand for regional utilities in the year 2014/15 to 2017/18 had steadily increased consecutively from 0.4289, 0.4350, 0.4420 to 0.4519 km^3, respectively (Water Utilities Regulatory Authority [EWURA], 2018). The trend implies that more water will be drawn from the same resources in order to meet demand; hence, the efficient use of scarce resources is vital. Therefore, strategies for the reduction of existing high levels of water loss are of high importance.

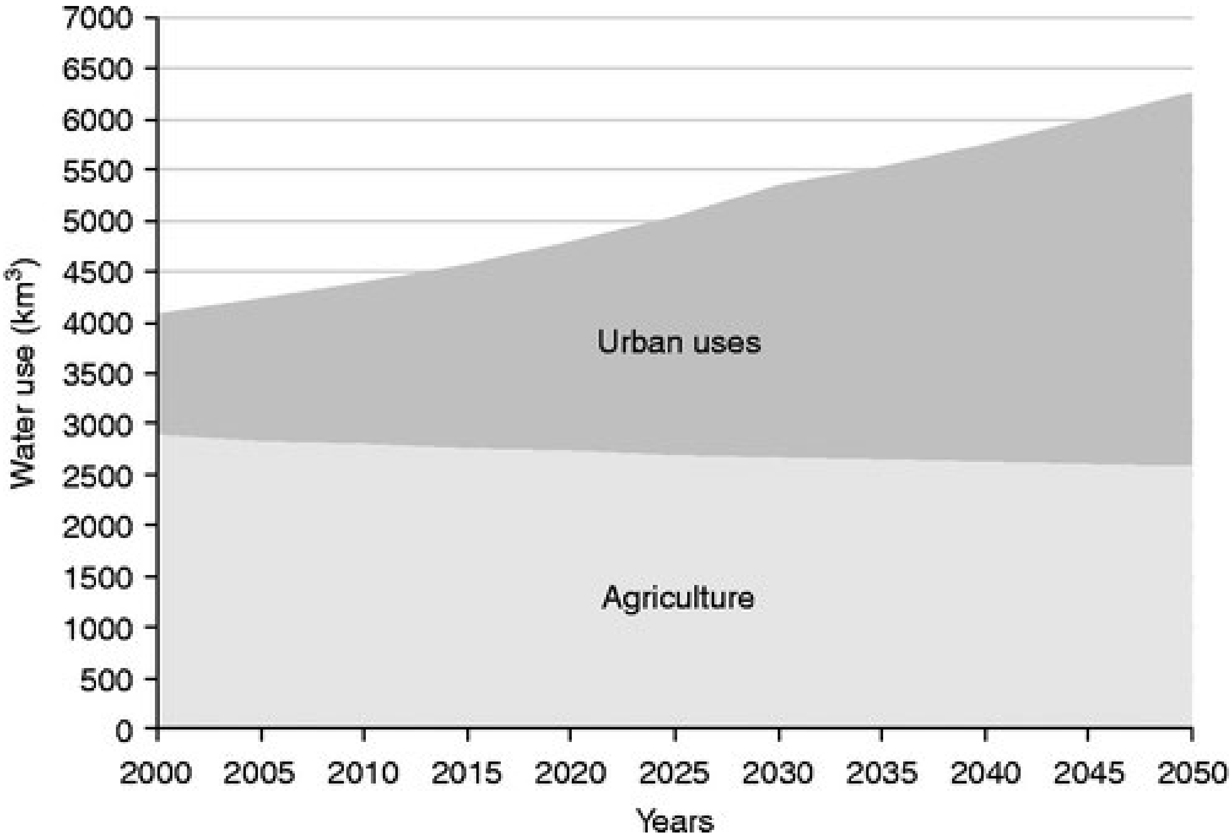


Figure 1: Trend of World water use from 2000 to 2050 (González-gómez *et al.*, 2011)

High levels of NRW indicate poor or inadequate, both water demand management and commercial practices (Zeraebruk *et al*., 2014). The problem of high NRW and low level of service continued for a quite long time for some cities in Africa. The average NRW approximated at 38% (Nairobi-Kenya), 51% (Lusaka-Zambia), 36% (Kampala-Uganda), and 32% (Johannesburg-South Africa) (Mwandosya, 2008). According to Tanzania Energy and Water Utilities Regulatory Authority (EWURA), Mwanza had 37%, while an average of 38% NRW was recorded for the whole country in the year 2016/17. As one of the highly growing cities, Mwanza is facing unplanned WDN, mainly in the informal areas that make up about 70% of the city. The percentage of NRW trend for five years from 2012/13 to 2016/17 was 42, 41, 41, 38, and 37 consecutively. The observed percentages are much higher than the EWURA benchmark of 20% (EWURA, 2017). It is also high compared to the threshold set for well-performing water utilities (23%) (Tynan & Kingdom, 2002). Reducing NRW is considered as the key to sustainable water supply provision, although it turned out to be challenging in its management. The NRW is also a global challenge where developing countries recorded about

50% average annual NRW according to the World Bank report of 2006. The high rates of NRW are due to various factors including unplanned WDN (Lee & Schwab, 2005; Mutikanga *et al.*, 2009). Several studies on NRW management have been conducted. However, information on high NRW for an unplanned WDN and fast-growing towns, especially in developing countries is still limited (National Audit Office [NAO], 2012; Mutikanga *et al.*, 2013; Gupta & Kulat, 2018).

There are some countries which are performing considerably well on management of non revenue water with less percentage of amount of water lost in the distribution network like Singapore, Denmark and Netherlands. For instance in Singapore where the NRW stands at 5% they emphasised appropriate water governance, strong political will and establishment of regulatory institutions in implementation of water management policies (Biswas & Tortajada, 2010; Luan, 2010). It is effective to implement engineering perspectives in reduction of NRW programs if the policies are suitable.

There is a wide application of various methods in assessment and simulation of Water Distribution Networks (WDN). However, they are not commonly used in engineering practice by water utilities in developing countries, including Tanzania. In bridging the gap between researchers and practitioners, the modelling approach used in this study can be applied to assess and solve high NRW and low levels of WDN performance. Furthermore, this study aims at investigating of WDN in fast-growing cities, unplanned expansion and simulate stochastic demand growth for sustainability of present and future service provision. Moreover, it gives awareness of possible consequences that may result if system designing and operations would not integrate stochastic towns expansion. Mwanza city was selected as a case study, as the area which faces high NRW problems as well as uneven water distribution. Although Dar es Salaam, Arusha and Mbeya have higher NRW than Mwanza but there were limitations to conduct research in the areas like being under study by other researchers and system were under major rehabilitation and construction. However, similar approach might be followed to solve the challenges facing the repective water utilities. The problem tackled through simulation by EPANET software, hydraulic analysis and field measurements applied to a sampled small demarcated area within Mwanza water network. The main goal is to maximize service delivery and minimize operating costs. Without WDN management, utilities may have little insight into the exact rate of water flowing to each part of the pipe network and the residual pressure at junctions. Thus, ensuring that consumers get water at adequate pressure, avoiding leakages and pipe bursts remains to be a major challenge.

1.2 Statement of the problem

The fast-growing towns, especially in developing countries, have poorly performing Water Distribution Networks (WDNs), characterized by high Non-revenue Water (NRW) and unreliable services (Zeraebruk *et al.*, 2014). Usually, the areas with high NRW experience water leakages in streets/ environment, under-registration of water meters, water theft, illegal connections and water network vandalism while there are people who receive water in extremely low or high flows and sometimes no water. For instance in Tanzania, the annual average NRW as a percentage of water production reported increased from 38 in 2017 to 41% in 2018 (EWURA, 2018). The percentages are still higher than the benchmark (20%). It also reported that high levels of NRW continued to be a global challenge (González-gómez *et al.*, 2011). Particularly Mwanza city, one of the fast growing-towns with an annual population growth rate of 3% (National Bureau of Statistics [NBS], 2012), and water services covering 74% of the total population has deficiencies in the level of services. Its networks comprised unplanned expansions, stochastic demand growth account for about 37% NRW (EWURA, 2017).

Uneffective management of the distribution system may worsen the financial and technical capacities of the utility. Thus, chances of further lowering the level of service to meet customer satisfaction due to high level of NRW is likely to occur. Therefore, assessment of the performance and simulation of the WDN is inevitable for system improvement and service sustainability. The proposed approach in addressing high NRW and uneven water distribution in a water network by incorporating unplanned and stochastic demand growth conditions to improve the level of performance of the system. Also, present and future simulations predict the improved performance that may help in decision making.

1.3 Rationale of the study

Many Sub-Saharan African cities have poor performing water distribution network with high NRW water than the recommended limits (Zeraebruk *et al.*, 2014). Due to urban sprawl and high rate of population growth there is unplanned expansion of the distribution network which lead to poorly performance of the system (Chirisa, 2008). One of the conditions that may contribute to high non revenue water is the presence of unplanned distribution network in an area (Lee & Schwab, 2005; Mutikanga *et al.*, 2009). However, the operations and maintenance of these systems in developing countries have not been given enough attention to ensure NRW is reduced to the recommended level (Schwartz & Sanga, 2010). A global water pressure forecasting indicates significant increments in water demand in urban areas (González-gómez *et al*., 2011). Thus, reducing NRW is considered as the key to sustainable water supply provision, although it turned out to be challenging in its management.

Hence, this study outlines measurements, simulation, performance analysis and improvements of water distribution system focusing on reduction of non revenue water in the unplanned distribution network. This will help water utilities in managing the NRW by analysing factors contributing to it and ways to reduce the percentage of the water loss.

1.4 Research objectives

1.4.1 General objective

The general objective of this study is to investigate the effect of the unplanned expansion of Water Distribution Network (WDN) on Non-revenue Water (NRW) and develop performance optimization scenarios for NRW reduction in Mwanza City.

1.4.2 Specific objectives

(i) To examine the effect of the existing WDN topology on NRW in a selected WDN.

(ii) To model and develop optimization scenarios for effective reduction of NRW of a selected WDN.

1.5 Research questions

(i) To what extent do the existing WDN contributed to NRW?

(ii) What is the optimization scenario for improving WDN performance of a selected WDN?

1.6 Significance of the study

The NRW is an essential parameter for prioritizing the improvement and management of a Water Distribution Network. Nevertheless, it has persisted high for some years. Proper management of water supply would improve the level of performance of the system and reduce NRW. When NRW is controlled and well managed, it increases the efficiency of the service delivery provided by the utility and the following can be achieved:

(i) Saving a substantial amount of water that is lost within the system. This would extend the life of water sources. For instance, Mwanza city may save from Lake Victoria water source part of 37% that is currently lost.

(ii) It can save costs for buying water treatment chemicals and energy for supplying extra water that is lost within the system.

(iii) More revenue to be generated by selling the lost water and reducing production expenses.

(iv) Level of service delivery improved through increase of revenue that assures the availability of more capital investments, operations expenditures, and staff incentives.

(v) It provides a stable system that prolongs the life span of the water facilities.

(vi) It increases customer satisfaction and reduces complaints.

(vii) The new population could be served by the water lost than looking for new sources. If you keep NRW at minimal level then you may save water and money for new projects.

(viii) The findings of this study shall increase understanding of hydraulic behaviours of the water distribution network in Mwanza city. It will create awareness on the status of percentage contribution of each component to the total NRW.

(ix) It encouraging the establishment of WDN models for other areas of the city for NRW measurements, analysis, reduction, control, and management.

(x) Informing and changing the paradigm of management of unplanned WDN and used as a guide for decision-making.

(xi) Increasing awareness on the consequences of ignoring the integration of uncertainties in demand growth for fast-growing towns during design and operations of the water systems.

1.7 Delianation of the study

The study focused on field measurements of the two essential paramenters in hydraulic performance analysis of the water distribution network. Other data from the water authority were collected to facilitate building a model of an existing distribution system. A case study was Mwanza city and a sampled District Metering Area (DMA) was detailed assessed to understand its behaviour compared to the entire network. The existing distribution network was investigated its contribution to the NRW. Also, optimization scenarios were developed to evaluate the best one which may improve performance of the system and reduce percentage of non-revenue water.

CHAPTER TWO

LITERATURE REVIEW

2.1 Water distribution networks

A Water Distribution Network (WDN) is a system that conveys water from the source to the point of use. It comprises pipes, valves, pumps, reservoirs, or tanks, as represented in Fig. 2 (United States of America- Environmental Protection Agency [USEPA], 2005). During the design of a WDN many parameters are considered, including; design life, per capita demand, average and peak demand, future expansion, allowable pressure, flowing rate and velocity. However, due to limited resources in developing countries, it is common that some parameters may be compromised which might lead to underperformance of WDNs. Thus, it is crucial to pursue water demand management during monitoring and control of the production at the source, treatment, storage and distribution operations including hydraulic and quality assessment. The performance of any WDN may be lowered by the emerging of organic WDN caused by urban sprawl due to political, social or economic factors. Management of water infrastructure in cities depends much on the increase of population and changes in spatial conditions (Christian *et al.*, 2017). Water Distribution Network is an essential part of water infrastructure which lives long and requires high capital investment; its effective management is vital. The analytical investigation of how the WDN performs by taking measurements including spatial pressure distribution, flow rate, WDN topology, water meter inaccuracy and unbilled water is important. This will help to establish potential factors contributing to NRW of unplanned WDNs.

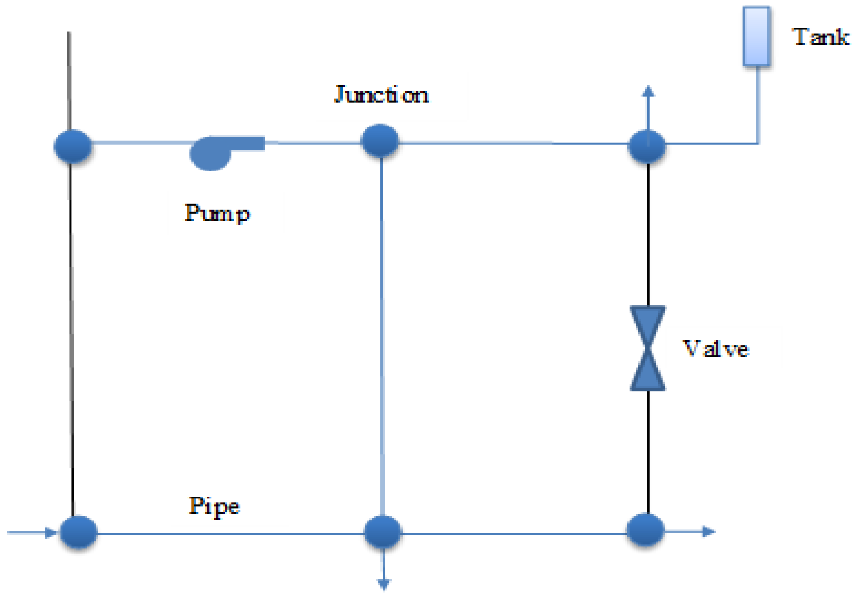


Figure 2: Typical elements of water systems in a model (USEPA, 2005)

Water Demand Management (WDM) is any action that will: (a) improve the quality or quantity of water required, (b) adjust a task or methodology that use less water and low quality, (c) moderate quality and quantity from source to disposal, (d) change peak to off-peak periods, and (d) raise capability of WDN to provide service during shortage supply (Brooks, 2006). Among the WDM policies established in the conservation and efficiency use of water, is leakage control and management which aims at the reduction of NRW (Stavenhagen *et al.*, 2018).

In common practice, the WDN has two main configurations namely dead-end and looped systems. Jolly *et al.* (2014) explained advantages and disadvantages of each type as follows: Advantages of a dead-end system of layout include: (a) The water flow and pressure in any location within the WDN can function precisely due to one-directional flow, so its design computations are simple; (b) less number of control valves are required compared with other types; (c) designing of pipes considers only the population intended to be served and thus, economical design is attained; and (d) it is simple in pipe-laying during construction. The disadvantages of dead-end systems include:

(i) In the case of repairing works in any section within the WDN, water supply to the entire portion beyond that point will be cut off. Thus, large portion of the distribution area will be affected, resulting in great inconvenience to the consumers of that area.

(ii) Due to many dead ends in the WDN, there are no free water circulations which result in stagnation of water. The stagnation may cause deterioration of water quality. In order to remove the stale water and deposited sediments in the system, there should be scouring valves installed at dead ends. This measure is costly for purchasing the valves and thrown water is lost during cleaning.

(iii) At the remote parts of the WDN, the system cannot maintain satisfactorily and desired pressure; and

(iv) Since there is only one side of supplying water, it is difficult to make water available during fire-fighting since all users will be depending on supply from one directional.

The other common layout is the looped system. The advantages of a looped system layout include: (a) The water circulates freely within the system hence no stagnation which could cause water quality degradation, (b) Water can be supplied from more than one direction due to network interconnection hence less head-loss, (c) Only a small area is affected during repairing as other areas may continue receiving water from other parts, and (d) During fire-fighting, much water may be diverted from other directions to the vulnerable part. The disadvantages of a looped system layout include: (a) More control valves are required to be installed hence increased costs, (b) More pipes are required for interconnecting the system, (c) Designing procedures are difficult and complicated in computing pipe sizes and working pressure, and (d) The system construction is more costly (Jolly *et al.*, 2014).

2.2 Pressure and velocity requirements in water distribution networks

During operations of WDNs, flow and pressure are the important parameters to be considered to ensure that the required hydraulic performance is met. Every customer should obtain the amount of water that can meet the demand at an adequate pressure. The design pressure should be divided into zones depending on the ground elevation or datum of a specific area in order to have proper pressure distribution. The status of pressure for each zone should be shared with relevant authorities including city planners so that they can integrate with their future planning of new areas.

The allowable best practice working pressure usually differs between various regions as seen in Table 1 (Ghorbanian *et al.*, 2016). According to the Tanzania Design Manual for Water Supply and Waste Water Disposal third version, URT Ministry of Water (2009), the minimum residual pressure at any point of consumption should not be less than 5 m. Requirements for fire condition is 15 m, maximum allowable pressure in the distribution network is 60 m, and velocity ranges are shown in Table 2. In the case where the difference of elevation of one pressure zone is higher than 10 m, an isolated sub-zones should be established by considering high, medium and low levels pressure and providing separate water supply mains.

Table 1: Pressure standards on review guidelines and regulations for various regions

	Minimum pressure (m) Condition				
Region	**During fire flow**	**During maximum hourly demand**	**During normal conditions**	**During all conditions**	**Maximum pressure (m)**
Canada					
British Columbia	14	28	-	-	70
Alberta	15	35	-	-	56
Saskatchewan	14	35	-	-	70
Halifax	15	28	-	-	63
Manitoba	14	21	-	-	N. S
Other provinces	14	-	28	-	70
USA					
Louisiana	-	-	-	10.5	N. S
Connecticut, Oklahoma, Delaware	-	-	-	17.5	
Michigan	14	-	24.5	-	
Other states	-	-	-	14	
United Kingdom and Wakes	-	-	-	10	N. S
Brazil	-	-	-	15	
Australia	20	-	-	-	
New Zealand	10	25	-	-	
South Africa	-	-	-	24	
Netherlands	-	-	-	20	
Hong Kong	-	-	-	20	

Ghorbanian *et al.* (2016); N.S = not specified

Table 2: Guide to velocities in water distribution network for various pipe diameters

Pipe Diameter range (mm)	Nominal Diameter (mm)	Velocity range (m/s)	Flow Min (L/s)	Flow Max (L/s)
Small DN 50-110	50	0.60 – 1.00	1	2
	65		2	3
	80		3	5
	100		5	8
	110		6	10
Medium DN 125-250	125	1.00 – 1.50	10	18
	150		18	30
	200		30	45
	225		40	60
	250		50	80
Large DN 300-500	300	1.20 – 2.00	80	140
	350		120	190
	400		150	250
	450		190	300
	500		250	400

Tanzania Design Manual (URT Ministry of Water, 2009)

2.3 Water balance and Non-revenue Water

The NRW is a summation of three components of input volume to a WDN such that real loss or leakage, apparent loss and unbilled authorized consumption as indicated in the IWA water balance table (Lambert & Hirner, 2000). The NRW is an important part of the water supplied to a system because of its economic impacts. This is because the already treated water lost is a wastage of resources, including treatment chemicals, energy for production and supply, and water as a scarce resource. There are challenges in many water utilities around the world on the definitions of terminologies in a water supply system. For example, water loss definition is mixed up with NRW, Leakage and Unaccounted for Water (UfW). In cognizant of this ambiguity, IWA established an international standard water balance as shown in Table 3 (Liemberger & Farley, 2004). The water balance table helped all regions in the world to have a common understanding and reasonable comparison of NRW within the water utilities.

Table 3: Standard IWA/AWWA international standard water balance

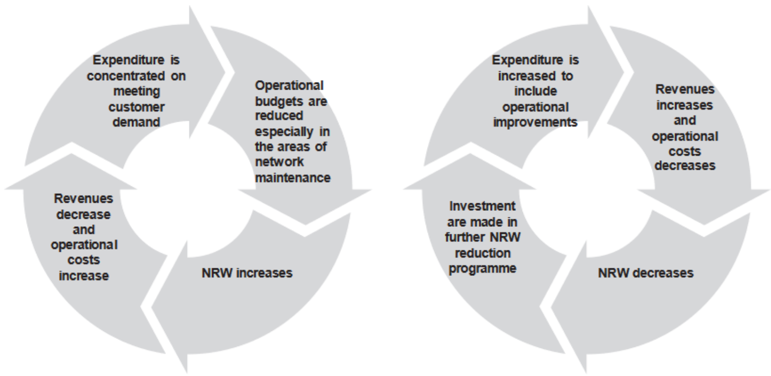
<table>
<tr><td rowspan="9">System Input Volume</td><td rowspan="4">Authorized Consumption</td><td rowspan="2">Billed Consumption</td><td>Billed metered consumption (including water exported)</td><td rowspan="2">Revenue Water</td></tr>
<tr><td>Billed unmetered consumption</td></tr>
<tr><td rowspan="2">Unbilled Consumption</td><td>Unbilled metered consumption</td><td rowspan="7">Non-Revenue Water (NRW)</td></tr>
<tr><td>Unbilled unmetered consumption</td></tr>
<tr><td rowspan="5">Water Losses</td><td rowspan="2">Apparent Loss</td><td>Unauthorized consumption</td></tr>
<tr><td>Customer metering inaccuracies and data mishandling</td></tr>
<tr><td rowspan="3">Real Losses</td><td>Leakage on transmission and/or distribution mains</td></tr>
<tr><td>Leakage and overflows at utility's storage tanks</td></tr>
<tr><td>Leakage on service connections (customers lines)</td></tr>
</table>

Farley and Trow (2003)

The system input volume is the total volume of water entering a defined system of a WDN at a specified time for various consumptions, including fire hydrant points, system flushing, customers, watering, and cleaning public facilities (source). For the sustainability of water operations, all this water is expected to be billed to understand the exact distribution of the water input. Authorized consumption is the volume of water consumed by users who are recognized by the respective authorities. Depending on operations arrangements of the respective authority, it can be metered whereby the water used is registered by a water meter and billed or unmetered whereby a flat rate or estimation use is computed and billed. Water loss is the remaining volume of water after subtracting authorised consumption which comprises of two main components of commercial or apparent losses and physical losses or leakages. Apparent loss is the volume of water lost through meter inaccuracies, billing errors, illegal consumptions, and theft. Real loss is the volume of water lost through leakages and overflows at different components of the WDN including pipes, tanks, service connections before customer water meters and reservoir or tank overflow.

2.4 Addressing Non-revenue Water

There are cost disadvantages on the performance of water utilities due to high NRW. Losses generated by leakages and pipe bursts increase operating costs and additional investments, while apparent losses reduce the revenue that would be gained by utilities. This has been illustrated in the vicious and virtuous circles in Fig. 3 (Msomi & Tarisayi, 2017). Increasing NRW reduces income to the utility that leads to its prioritization in satisfying customers than maintenance of the network, which increases production costs. However, when more investment is put on NRW management, revenue will increase and the utility will be able to cover costs for maintenance and operations of systems. Transforming from vicious circle to virtuous circle is the main challenge the water utilities should attain in order to reduce operating costs and increase revenues.



(a) (b)

Figure 3: Vicious and virtuous circles of non-revenue water (Msomi & Tarisayi, 2017)

There are no satisfactory results achieved on reducing NRW, although most water utilities always prepare strategies to combat. The following main points cause failure to NRW management:

(i) Managing NRW requires the commitment of every department in the utility.

(ii) Non-Revenue Water should not be solved as an isolated technical problem, rather linked to financial, asset management, commercial aspect and other more factors.

(iii) Non-Revenue Water programs might be considered as a project of indirect impact on revenue increase, which may cause more investment be given to new projects.

(iv) Wrong understanding of NRW magnitude and proper approach of its reduction.

Among drivers of NRW, some cannot be controlled by the utility and some are under their control (Berg, 2015). Two main factors that are out of the control of the utilities are network topology and population density per kilometre of network. These are mainly caused by urban sprawl and settlement planning in fast-growing towns. Other factors include low opportunity costs of water losses, high repair costs of water losses and utility's country environment. The reduction of NRW is key to improving utility performance but does not go hand in hand with utility performance improvement tools. Let us say energy reduction programs can increase utility revenue, but improved energy efficiency will also reduce the opportunity costs of NRW.

2.5 Leakage assessment in a water distribution network

In general, there are three methods of assessment of the water loss in a WDN i.e. Bursts and Background Estimates (BABE), Minimum Night Flow (MNF) and Top-Down Water Balance. The first two are called the bottom-up approach which may be applied in assessing and quantifying real losses in WDN (Xin *et al.*, 2014; Mazzolani *et al.*, 2016). The top-down approach requires presence of national standard water balance methodology showing the assessment of all the components of consumptions (Xin *et al.*, 2014). Due to that limitation the second was adopted in this study. The bottom-up approach by MNF was adopted rather than computational methodology from calibration due to limited real-time data available from Supervisory Control and Data Acquisition (SCADA) system (flow, pressure, head losses) (Cheung *et al.*, 2010).

Every method has its limitations depending on the specific network and local conditions. According to a review by Al-washali *et al.* (2016), the MNF analysis is probably the best method and provides more meaningful results because it is estimated based on actual data measured. Three items which must be taken into consideration are measurement tools, night consumptions (consider different use habit) and average pressure. The analysis shown in Fig. 4 is recommended for the small part of hydraulic demarcated distribution network or DMAs. The leakage is numerically computed by using mathematical equations and this value is then subtracted from the total water loss to obtain the apparent loss. The flow and pressure measurements are conducted for 24 hours where the minimum night flow and average system pressure are determined. During the night hours, flow is expected to be minimum when leakages are considered as the main consumptions since almost everybody is expected to be sleeping and the pressure within the system is expected to be maximum at this time.

The Burst and Background Estimates method uses relatively the available system data and is more practised in developed countries. It analyzes the real loss into smaller components, so it helps the responsible authority to understand in detail the nature and type of leakages that occur within the system. However, due to specific cases, assumptions put in establishing is 'model' it cannot be used internationally as every network has different characteristics. Furthermore, it only suits water utilities that carry out regular leakage control, which is not usually the case especially by developing countries. It is recommended to be used as a supplementary activity for breaking down the leakage components. Leakages mainly occur either as pipe bursts or background leakages. It is easier to identify bursts in the transmission lines and distribution systems as everybody can see water flowing on ground and report to the respective authorities. A quick response can be taken to fix the damaged infrastructure and water supply interruption. The total volume of leakage by bursts is calculated by multiplying the number of reported bursts, leakage flow rate, and average leak duration. The estimation of leak flow rates may also be done for mainlines 240 l/h/m pressure and 120 l/h/m pressure for reported and unreported bursts respectively. For service connections it is 32 l/h/m pressure for both reported and unreported bursts. For background leakage, as developed by IWA water task force the estimate of leakage at the mains, service connections to property boundaries and customer meter are 9.6, 0.6 and 16 litres per kilometre per day per meter of pressure, respectively. This type of leakage is difficult to be recognized as it occurs underground and needs special equipment for detection (Lambert, 2002).

The top-down approach estimates the apparent loss after calculation of the authorized consumption. The real loss is computed by subtracting the apparent loss from the total water loss. According to a review by Al-washali *et al.* (2016), part of the authorized consumption which is unbilled is assumed in the range of 0.5% to 1.25% of the input volume. He also suggested that the unauthorized consumption (part of apparent loss) be in the range of 0.25% to s1% of the billed water for developed countries and about 10% for developing countries. For meter inaccuracies, the estimates are based on the data from regular meter testing results of different flow rates for typical customer consumptions and related manuals. Data errors can be estimated from the records of the data billing trend of previous months.

As explained by Makaya (2017) computation of real loss performance indicators; Infrastructure Leakage Index (ILI), Unavoidable Annual Real Loss (UARL) and Apparent Loss Index (ALI) within water authorities could also be used as one of the comparison indicators of system efficiency. However, almost all water utilities except for the United Kingdom use percentage of the input volume as an indicator. The ILI is a measure of the level of the management in controlling real loss at current operating pressure (repairs, pipelines and asset management, active leakage control) (Liemberger & Farley, 2004). It describes the technical performance of the real loss without taking into account the investment and resource costs. It is computed by taking the ratio of Current Annual Real Losses (CARL) in litres per day per connection to UARL in litres per day per connection. The CARL is always increasing with the age of water distribution and ALI calculated by taking the ratio of apparent loss to 5% of billed authorized consumption.

$$UARL(litres/day) = \left(18 * L_m + 0.8 * N_c + 25 * L_p\right) * P \quad 1$$

where L_m is the length of the main pipes (km); N_c is the number of service connections; L_p is the total length of service pipelines (m) and P is the average operating pressure (m).

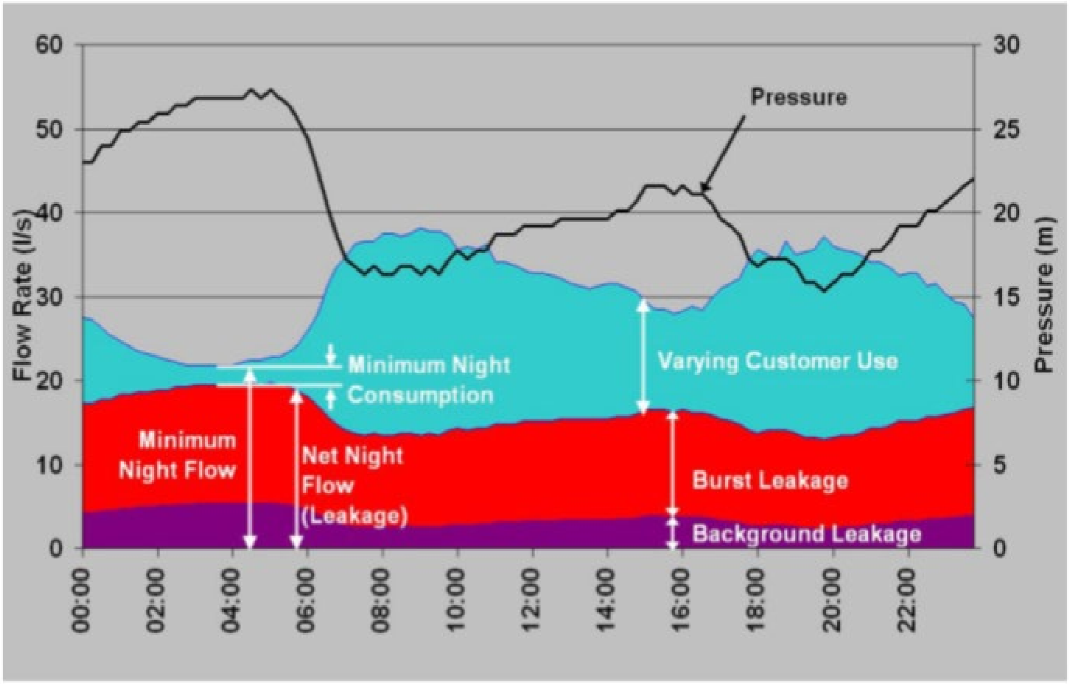


Figure 4: Variation of flow indicating MNF, pressure and leakage in a DMA (Al-washali *et al.*, 2016)

2.6 Modelling of a water distribution network

The world development in technology brings challenges to improving operations performance of various sectors. Mathematical models are one of the technologies available in water industry. Experienced staff usually adjusts control components like valves and pumps during operations and general management of water systems to ensure customer satisfaction. Modelling is a pre-emptive management approach. For instance, hydraulic modelling gives insight assessment of performance of water distribution networks and their functions. Model simulation extends from analysis of present to future system conditions. It gives mathematically replicate capability to the dynamics of WDN by solving conservation of mass and energy conservation Equations (Bhoyar & Mane, 2017). Also, new and rehabilitation of water systems can be designed by using hydraulic models. The following analysis can be done by modelling:

(i) Extended periodic

(ii) Steady-state

(iii) Hydraulic transient

(iv) Scenario development

(v) Optimization of pressure and operational

(vi) Emergency and existing supply

(vii) Fire flow

Modelling can apply optimization approach to search the best solution to an identified problem for several iterations and variables. It finds the highest feasible performance under given constraints. The more a model replicate the site conditions, the more realistic to design and management attained. It should be clear that modelling provides information and not decisions. However, modelling of a water distribution network has challenges on:

(a) Accuracy of input data and design approach

(b) Correct understanding of required outputs

(c) Availability of software to be used

(d) Time availability

Most water utilities in Tanzania do not keep records of their assets in computers, including water infrastructure. For instance, most of the as-built drawings are found in hard copies, and mostly the long-time constructed infrastructure operated merely by experienced old staff members. This is a big challenge in the management of the operations and maintenance of the WDN. Some water utilities have started using the Georgraphical Information System (GIS) technology to store the WDN maps, but the challenge still is the exact location of some of old pipes and valves. This change is a process and needs regular updating of the GIS maps to ensure the reality of the maps compared to field situations. Similarly, WDN modelling is almost not done by water utilities, which makes the WDN expansions to follow the population settlements without prior planning. The impact of random expansion on the entire network should be checked rather than considering only a small part while they use the same source and transmission pipelines. The increase in demand and topology change of the network directly affects the entire system. This can be done with the help of modelling tools and approaches.

2.6.1 The EPANET 2.0 modelling software

The perpetual increase in water demand due to population growth requires a properly modelled WDN applying the computing technologies, including hydraulic modelling (Ramesh *et al.*, 2012; Sonaje & Joshi, 2015). Significant reviews are done in public domain models like EPANET, Branch and Loop; and commercial models like Aquis, WaterGEMS and WaterCAD. The EPANET is a widely free applied simulation model for analysis of water hydraulic behaviours in WDN (Georgescu & Georgescu, 2012). This open source software has more advantages as its source code is freely available and learning materials do not require permission of vendors. A framework for decision-makers in extension and management of supply network and location of new facilities usually should be provided based on the models.

Standard hydraulic models mainly have two essential components in order to work as a system, i.e. junctions or nodes and links or pipes. These two components are linked to each other to make connectivity and allow for the flow of water within the system. Together with other supporting components in Table 4, simulation of EPANET modelling software can be done. The EPANET uses two principle equations in characterizing the hydraulic state of the WDN in a pipe between junctions i (hydraulic loss or energy conservations) and j (continuity or mass conservation) as shown Equations 2 and 3 respectively.

$$H_i - H_j = h_{ij} = rQ_{ij}^n + mQ_{ij}^2 \qquad 2$$

$$\sum Q_{ij} - D_i = 0 \qquad 3$$

where H = nodal head, h = head loss, r = resistance coefficient, Q = flow rate to node i, n = flow exponent, and m = minor loss coefficient and Di is the flow demand at node i. Continuity equation uses the law of mass conservation which states that, in the absence of nuclear reaction, the matter can neither be created nor destroyed. It takes into account the volume of water flowing in its pathways (pipes) and junctions for a specific time. The principle is that at any junction, the mass flow rate of water entering (M_{IN}) should be equal to the mass flow rate of water leaving (M_{OUT}) see Fig. 5.

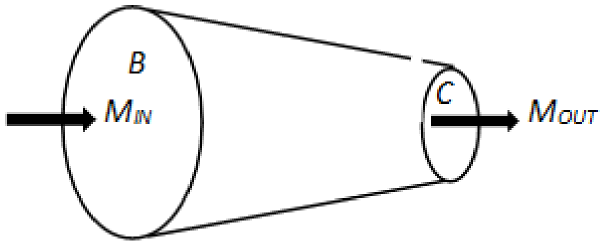


Figure 5: Principle of the low conservation of mass of water flowing from B to C

Mathematically:

$M_{IN} = M_{OUT}$ But $M = \rho V$

Thus, for a rate of the mass of water from various pipelines in a junction with same density,

$M = \sum_{IN} \rho V = \sum_{OUT} \rho V$ $but\ V = A * v$

$$\sum_{IN} \rho A_B v_B - \sum_{OUT} \rho A_c v_C$$

$$Q = A_B v_B = A_C v_C \quad 4$$

where Q is the volumetric flow rate/ discharge; ρ is the density of a fluid, V is the volume of fluid, A is the cross-sectional area of a pipe; v is the velocity of the fluid.

The energy Equation applies the law of conservation of energy by considering that, during the transfer of fluid there are some energy losses due to frictions, joints and fittings or bends. So, the energy at point A should be equal to the energy at point B, plus all the energy lost during fluid conveyance. Among the three energies contained i.e. thermal, chemical and mechanical, the latter is the applicable energy in this scenario. The mechanical energy in the flowing water comprises three components of pressure, potential and kinetic energy. According to Bernoulli's principle, the energy Equation of flow of water at a section regarding the datum line is the summation of elevation of pipe centreline from the datum, pressure head (piezometers) and velocity head due to average velocity of the flow at the section. This is the fundamental Equation for almost all hydraulic calculations. Figure 6 shows the energy conservation Equation.

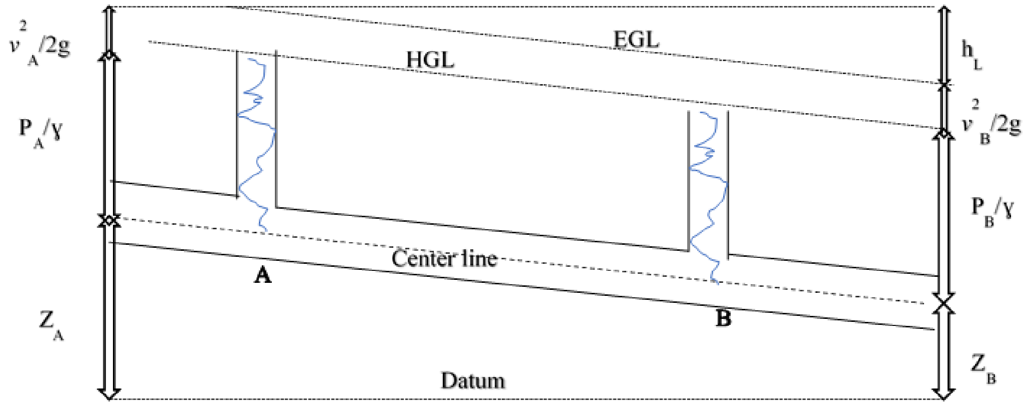


Figure 6: Variation in energy as a unit mass of water flowing from point A to point B

Energy = potential (P_A/γ*) + pressure (*ρgh *)+ kinetic (*$\rho\frac{v^2}{2}$*)*

$$E = \rho gz + p + \rho\frac{v^2}{2}$$

The expression is stated in terms of height or head of water for conveniences by diving each term with ρg:

$$E = z + h + \frac{v^2}{2g}$$

During water flow in a system, the total energy is obtained by adding any energy imparted to the water for boosting and subtraction of any energy lost during transportation of the water between two sections.

$$z_A + h_A + \frac{v_A^2}{2g} + h_p = z_B + h_B + \frac{v_B^2}{2g} + h_{L,A-B} \qquad 5$$

Where: EGL is a linear value (Nm/N) that represent the energy at each section and slopes in the direction of flow except where there is another energy imparted by mechanical device like pump; HGL is the hydraulic grade line that maintains the water levels in tubes at elevation; v is the average velocity of flow; p is the pressure of water; z is a height above the given datum; $h_{L,A-B}$ is the energy head loss between two points A and B; h_p is the energy head imparted to the water by the pump.

The frictional head loss through a pipeline is the important component in the fluid flowing computations because of its role in maintaining the flow in the pipe. It is part of the total energy which is converted into thermal energy due to friction, fluid viscosity, and turbulence. Such conversion is a loss of energy which is known as head loss expressed in the equation below:

$$h_L = \frac{f*L*v^2}{2*g*D}$$

where f is the friction factor (not easily computable and depends on pipe and flow characteristics), L is the length of pipeline section in m, D is the diameter in m.

The most popular head loss equation is the Hazen-Williams equations (Equation 6):

$$v = 0.85 * C_{HW} * R^{0.63} * S^{0.54}$$

$$h_L = \frac{10.67*L*Q^{1.85}}{C_{HW}^{1.85}*D^{4.87}} \qquad 6$$

where v is the average velocity in m/s, C_{HW} is the Hazen-Williams coefficient, R is the hydraulic radius (Area/Perimeter (wetted)) in m, S is the energy slope (h_L/L), Q is the flow rate (m^3/s).

Table 4: The EPANET required input data and output

Component	Input parameters and Descriptions	Source	Output parameters
Junctions	▪ Elevation amsl ▪ Water demand (rate of withdrawal from the network as per consumption pattern)	DEM billing database	▪ hydraulic head (internal energy per unit weight of fluid) ▪ pressure
Reservoirs	▪ hydraulic head	WSSA records and DEM	no computed output
Tanks	▪ bottom elevation (where water level is zero) ▪ diameter (or shape if non-cylindrical) ▪ initial, minimum and maximum water levels	WSSA records	hydraulic head (water surface elevation)
Pipes	▪ start and end nodes ▪ diameter ▪ length ▪ roughness coefficient (for determining head-loss) ▪ status (open, closed, or contains a check valve).	WSSA records and digitized map	▪ flow rate ▪ velocity ▪ head-loss ▪ Darcy friction factor
Valves	▪ start and end nodes ▪ diameter ▪ setting ▪ status	WSSA records	flow rate and head-loss
Inlet	▪ System Input volume (the volume of water input to a WDN)	Field measurement and WSSA records	no computed output

2.6.2 Emitter functions in modelling leakages

Emitters are the devices used in the EPANET to model water discharge to the atmosphere at junctions through nozzle or orifice. The discharge rate usually varies with residual pressure head existing at the respective junction. The principle mathematical expression according to the EPANET manual, is shown in Equation 3. This is the property in the nodes that were introduced to model the flow discharge to the atmosphere, which was used to simulate leakages in the WDN. Its functions depend on the pressure variations in the nodes. A recent report shows that this is the best method for representation of leakages in the simulation of the WDN model whereby it is using leakage as a dependent parameter to the pressure in the system (Sebbagh *et al.*, 2018). The actual demand obtained in the model results is equivalent to ordinal consumptions by customers plus the leakages (flowing water in emitter). Leakage distribution based on the junctions and pipe lengths is computed based on Equations 7 and 8 (Sebbagh *et al.*, 2018):

$$q = C\ H^{N1} \tag{7}$$

$$\frac{q_1}{q_2} = \left(\frac{H_1}{H_2}\right)^{N1} \tag{8}$$

where q is the leakage flow rate; C is the discharge or leakage coefficient; H is the pressure head; $N1$ is the flow exponent; q_1 and q_2 are leakage flow rate before and after pressure reducing system pressure; H_1 and H_2 are the corresponding system pressure heads respectively.

The value of leakage coefficient has less impact compared to flow exponent, which is used to describe the relationship of pressure and leakage in the system. So leakage flow exponent has high divergence of values because the leakage is not fixed, rather it varies with pressure (Fox *et al.*, 2016). The flow exponent can be obtained by measuring the leakage flow rate and corresponding pressure heads during minimum night flow conditions with system pressure variations. In Equation 8, the value of $N1$ can be estimated by the expressed pressure-leakage relationship.

Walski *et al.* (2006) cited Mays (1994) that the value of flow exponent is 0.5, 1.5 and 2.5 for the fixed area, pressure-dependent, and longitudinal leakages respectively as shown in Fig. 7. Through 50 tests, the observed values were ranging from 0.52 to 2.79 depending on flexibility of pipe materials. For the flow exponent of 0.5, when half the pressure reduces leakage will reduce by 65% and 2.5 will reduce by 82%. According to Cheung *et al*. (2010), many countries have conducted experiments to determine the value of flow exponent in a WDN and found the N1 value varies depending on pipe materials and failure mode. For metal pipes, it is 0.5, while for plastic pipe materials, it ranges between 1.5 and 2.5 with higher value for more flexible plastic pipes. Van (2007) demonstrated the outshot of flow exponent on the leakage flow rate, as indicated in Fig 10. When the pressure at the leakage point is reduced by half, then the flow exponent 0.5, 1.0, and 2.5 will be reduced by 29%, 50% and 82% respectively, where H is the values of pressure at the leakage points and q is the leakage flow rate. As per Hanusch *et al.* (2007), the flow exponent for cast iron pipes was 0.85; for PE was 1.5 and 0.9 for asbestos. The discharge coefficient of different networks varies depending on the geometry of orifices area, including pipe materials, leak openings, and pressure head. For a completely turbulent flow on circular openings, the discharge coefficient is 0.6, while for transitional flow, the coefficient is higher. Nevertheless, the average coefficient of discharge ranges from 0.5 to 0.8, and a mean value of 0.65 was considered for a coefficient of discharge, which demonstrated to be normally distributed. Trout (1986) derived value of discharge coefficient of 0.6 from his experimental results of which ranged from 0.438 to 0.673.

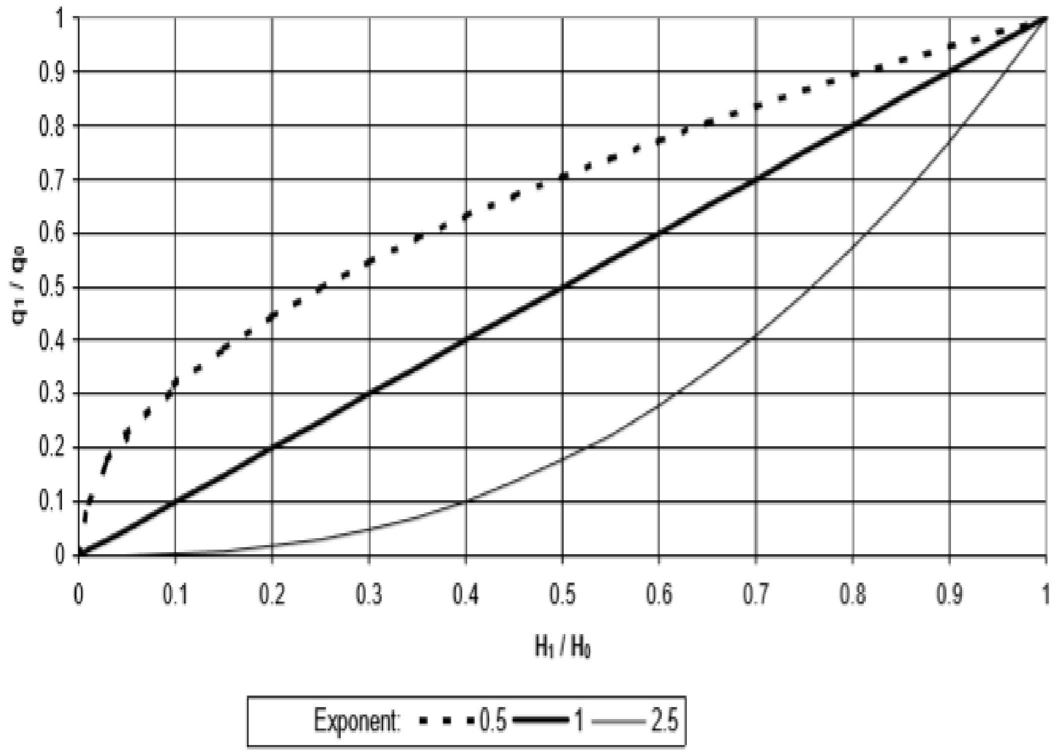


Figure 7: The relationship of the leakage flow rate and pressure to different flow exponents

2.6.3 Calibration of a water network model

Calibration is an important part of any modelling because it explains the relationship between the variables obtained during simulation and the data set obtained by field measurements. It is a process of simulating the WDN model to mimic the real water network by:

(i) Adding the map of WDN in the EPANET model and simulate.

(ii) Compare the observed data (flow and pressure) with the results of WDN simulation.

(iii) Calibrate by considering values, location and time of measurements for the average pressure and flow field data (Cheung *et al.*, 2010).

Correlation tells about the direction and strength of a linear relationship shared between two quantitative variables. For modelling, it explains the relationship between variables obtained during simulation and data set obtained by field measurements. In describing the relationship of two variables of any function, a time plot has a response variable. The variable measured the outcome of a study in y-axis and explanatory variable in x-axis, which explains the outcome of a study. In a scatter plot, there is no need to show time in the x-axis, unlike for the time plot. In this case, the two quantitative variables are considered where the explanatory variable in x-axis is the independent variable and the response variable in the y-axis is the dependent variable. Sometimes it happens that there is no explanatory variable, and the response variable is a function, so it does not matter where the two variables are plotted in either axis. This is commonly seen where there is a comparison of two unrelated variables or events.

When determining the correlation of a function, explanatory variables and response variables are not necessary, it is completely symmetric. So, correlation can be expressed by using the scatter plots, and it measures the direction and strength of the variables. The positive or negative value of a correlation tells about the direction of the slope for a data set. If the data set of the variables follows exactly the straight line in either direction, a perfect correlation that indicates good linear relationship is obtained. Correlation, "r" measures the strength of the linear relationship between the variables, where the strength increases as "r" gets closer to one. In absolute value, if the value of "r" is zero, it explains that there is no linear relationship whatsoever between the two variables and it means there is complete absence of a correlation. Correlation can be calculated using Equation 9:

$$r = \frac{1}{(n-1)S_x S_y} \sum (x_i - \bar{x})\,(y_i - \bar{y}) \tag{9}$$

where n is the number of data set; S_x is the standard deviation of variables x; S_y is the standard deviation of the variables y; $\bar{x}$ is the mean of the variables x and $\bar{y}$ is mean of the variables y.

The correlation coefficient may be squared to obtain the coefficient of determination which expresses the percentage of variance in one variable that is explained by the other variable. It is very important to understand which values of the correlation coefficient are strong by checking the value of coefficient of determination, r^2. For example, if $r = 0.5$, it means only 25% of the variance of one variable is explained by the other variable, and if $r = 0.7$, the variance has not reached even 50%. Mathematically, for a strong relationship between the two variables, the value of r should be at least 0.8 because it explains more than 50% of the variance in one variable with the other. For the two variables, if "y" is the dependent variable and "x" is the independent variable, the relationship between the two is described as regression of y on x. The regression phenomenon is simply expressing that the average value of y is a function of x and can be represented by the regression Equation 10. It shows how much value of y changes with the given value of x changes, unlike correlation that does not imply causation. Equation 10 helps to construct a regression line on a scattered plot. In correlation, we measure the direction and strength of the relationship between the two quantitative variables while in a regression there is an actual line drawn on the graph (regression line) representing the parallel of data best fit.

$$\hat{y} = b_0 + b_1 x$$
10

Where $b_0 = \bar{y} - b_1\bar{x}$ and $b_1 = r\,\frac{s_y}{s_x}$

2.6.4 Pressure reducing valves in the Water Network

These are valves used in protecting a DMA water network from bursts and leakages by reducing excessive pressure in the system. The Pressure-Reducing Valves (PRV) setting may be operated inactive, open or closed modes. The active mode operates when pressure at the downstream of the link is less than upstream pressure but higher than the PRV pressure setting. In this mode the PRV pressure is set at a pressure equal to the downstream pressure (Simpson, 1999). A PRV is said to be in an open mode if the upstream and downstream pressures are less than the setting PRV and it has no purpose to serve. For analysis, link is treated as open with zero head loss. A closed mode of PRV occurs when the pressure at downstream becomes more than the PRV pressure setting, or downstream pressure exceeds upstream pressure. For analysis, the link is considered to be eliminated and the corresponding entry in the coefficient matrix is set equal to zero. A PRV maintains constant pre-set pressure downstream regardless of the upstream pressure. Thus, several PRVs together can reduce excess pressure. The two exceptional behaviours of the PRV are (a) If pressure becomes less than the setting PRV pressure, it becomes inoperative and has no effect inflow, except head loss; (b) If PRV is bypassed and the downstream exceeds PRV pressure setting then it acts as check valve preventing backflow.

2.6.5 Polynomial functions (Exponentials and Logarithms)

The logarithms can be applied in non-linear trend data to change the curved functions of mathematical relationships to linear equations, and simplify solving equations by linear programming. The straight-line plotted graph from the exponential data which were changed to logarithmic data can be used to approximate the equation of that line graph. In principle, the two graphs of exponential and logarithms are the same; the only difference is scale size. From the exponential function logs are taken to both sides and split using laws of logs. The variables in y-axis and x-axis have log values, one constant intercepts (log y-axis), and in computing this constant, the antilog is computed. The power value is a slope that indicates the direction of the trend data whereby upwards direction will give positive value and downwards a negative slope value. The line of best fit will help to estimate the values of the two constants as shown in Fig. 8.

$$y = ax^b$$

$$\log(y) = \log(ax^b)$$

$$\log(y) = \log(a) + b\log(x)$$

$$Y = MX + C$$

$$Y = \log(y)\,;\ C = \log(a)\,; M = b\,; X = \log(x)$$

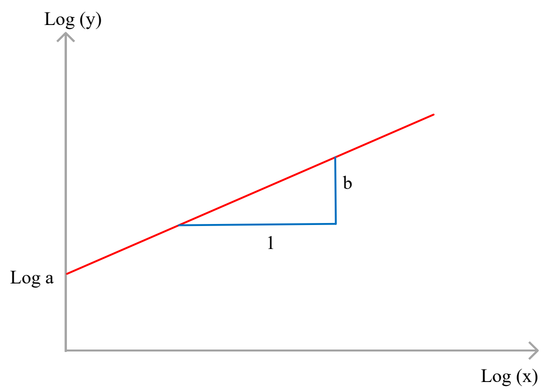


Figure 8: The logarithm plot of log y VS log x

2.7 Reliability of a water system

The WDN reliability assessment has gained importance and much attention due to high percentage of investment costs of about 70% compared to other water structures in a system. In this context, the reliability of WDN has received attention in order to understand and ensure that adequate and sustainable water supply services are provided. Reliability can generally be defined as probability of the system to work within the stated thresholds, time and conditions. Four interrelated factors can be derived:

(i) Expressing the reliability as a function of probability;

(ii) Requirement of the system to adequately perform its functions;

(iii) The duration which the system performs adequately; and

(iv) Specifying the operating conditions. High reliability of a component or system is important because it assures that the system meets its objectives. For this reason, the measuring level at which a system performs is therefore required for proper management of the system.

Water network reliability should be defined in what situation the system performs or not, both spatially and temporal. For example, it might not be of significant concern if there is no water to customers at 0300 h for one hour, but it can aggravate the situation if the customers fail to get water for the same duration at 0800 h. Also, intermittent water supply to residential areas might be of less concern compared to unplanned outage of water supply to a hospital. The reliability can also be used in accounting for the occurrence of a reduced period of services when water is supplied but at flow rate and pressure less than the required. The water operators have interest in mechanical and hydraulic types of reliabilities. Mechanical reliability refers to operability of the system or component, while hydraulic is measured by consistently providing the services at the required amount, pressure, time and location (Shinstine *et al.*, 2002; Gheisi *et al.*, 2016).

2.8 Sustainability index (SI) of the Water Distribution Network

The effectiveness of a WDN can be estimated through performance evaluation by computing its sustainability index (SI). The performance criteria are determined by three sustainability indices i.e. reliability, resiliency and vulnerability Aydin *et al.* (2014). According to Aydin *et al.* (2014), these three criteria can be used to evaluate and measure the level of probability of the system to fail (reliability), how quickly it can recover from the failure (resiliency) and what extent of the consequences of failure magnitude or time of unexpectable modes (vulnerability). The SI ranges from 0 to 1 from the lowest to the highest sustainability respectively. Borzì *et al.* (2018) used the ranges of values in Table 5 to describe the levels of SI of a WDN. To evaluate the performance of the system, we need to establish a mathematical relationship that will differentiate satisfactory from unsatisfactory performance of the WDN. For this particular study, the minimum and maximum pressure in nodes are considered to range between 5 m to 60 m respectively. The thresholds were taken from the Tanzania water design manual (URT Ministry of Water, 2009). The satisfactory and unsatisfactory modes can mathematically be expressed, as seen in Equation 11 (Aydin *et al.*, 2014).

$$P_{j,t} = \begin{cases} unsatisfactory(0) P_{j,t} \leq P_{min}; P_{j,t} \geq P_{max} \\ satisfactory(1) P_{j,t} \geq P_{min};\ P_{j,t} \leq P_{max} \end{cases} \quad 11$$

where $P_{j,t}$ is the pressure at junction j at time t, P_{min} and P_{max} is the minimum, and maximum pressure heads thresholds.

For each junction, Borzì *et al.* (2018) described the definitions mathematically to the three indices (reliability, resiliency and vulnerability).

Reliability (REL): $REL_{k,j} = \frac{number\ of\ times\ satisfactory\ occurs}{total\ number\ of\ time\ steps}$

Resiliency (RES): $RES_{k,j} = \frac{number\ of\ times\ satisfactory\ follows\ unsatisfactory}{total\ number\ of\ time\ unsatisfactory\ occurs}$

Vulnerability (VUL): $VUL_{k,j} = \frac{summation\ of\ unsatisfactory\ values}{total\ number\ of\ values}$

Sustainability Index (SI): $SI_{k,j} = [REL_{kj} * RES_{k,i} * \left(1 - VUL_{k,j}\right)]^{1/3}$

where k is the pressure at junction j

For each zone: $SI_{k,i} = \frac{\sum_{j=1}^{N_i} D_{i,j} * SI_{k,i,j}}{\sum_{j=1}^{N_i} D_{i,j}}$

Where $SI_{k,i}$ is the SI in terms of pressure in zone i, N_i is the total number of junctions in zone i, $SI_{k,i,j}$ is the SI in terms of pressure at junction j in zone i, $D_{i,j}$ is the daily demand of junction j in zone i.

Table 5: Sustainability ndex ranges

SI Range	State
0.00 – 0.25	Unacceptable
0.25 – 0.50	Moderate
0.50 – 0.75	Acceptable
0.75 – 1.00	Ideal

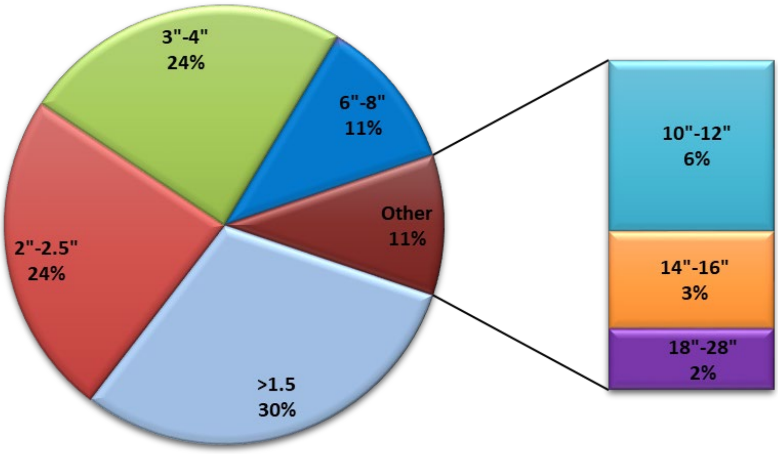
The major target in any infrastructure is to ensure its sustaibnability for robust and resilient system which cater for future demand. The performance analysis of the water distribution network and its modelling is important to ensure important data are available that may help in decision making and improving the system. The NRW which is one of the challenges facing fast-growing cities in developing countries require attention and more research to understand the main drivers in unplanned distribution network. Currently, there is no enough information provided and NRW management is remaining the main challenge in most water utilities. It is important to determine spatial and temporal changes of network topology and its contribution to system performance. Also, various scenarios need to be established to find the best one that can fit to similar nature of the network.

CHAPTER THREE

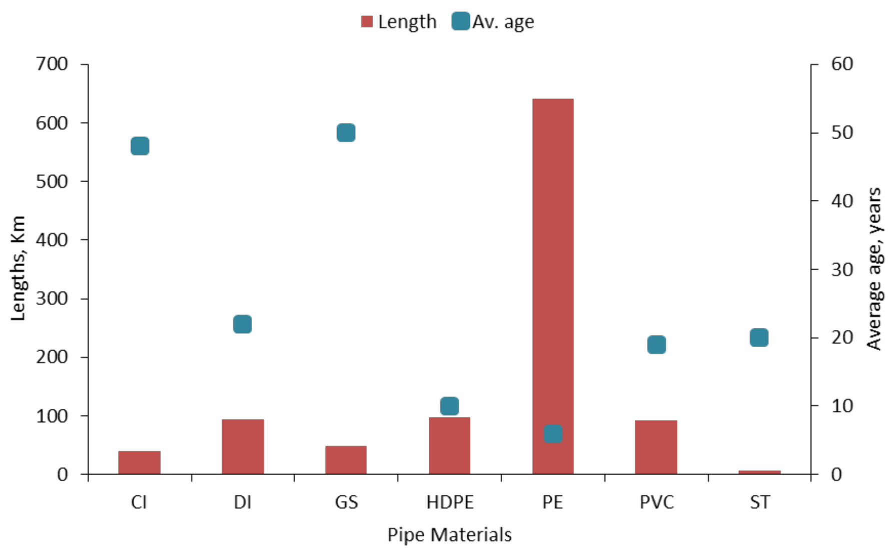
MATERIALS AND METHODS

3.1 Study area

Mwanza City is the second-largest urban center in Tanzania after Dar es Salaam, it is estimated to have a total population of 106 000 in 2019 (NBS, 2012). It comprises of two districts of Nyamagana and Ilemela. Geographically, Mwanza City lies along the southern shores of Lake Victoria on the North West of Tanzania between latitudes 2.15^{o} - 2.45^{o} South and longitudes 32.45^{o} - 33.00^{o} East. It has an area of 1337 km^2 out of which 900 km^2 is underwater and 437 km^2 is dry land. The body responsible for management of water infrastructure and supply in Mwanza City is Mwanza Urban Water and Sanitation Authority (MWAUWASA). As on 30th June 2018, the Mwanza City WDN had a total length of 789 km with pipe sizes range between 1.5”-28” (Fig. 9a), water customer base 74 314 (74% of total population served), the installed daily water production capacity 108 000 m^3 and daily water demand 116 575 m^3. As shown in Fig. 9b, most of the pipes are Polyethylene pipes (PE), after which High-density PE (HDPE) and Ductile Iron (DI) follow. The Galvanised Steel (GS) and Cast Iron (CI) are the oldest with an average age of around 50 years. The total annual water produced in 2017/18 was 73 233 m^3 which is lower than production capacity and water demand due to stoppage of water production caused by power outage Water is pumped from the main water intake located at Lake Victoria to the conventional treatment plant at Capri-point hill. It is conveyed through piped networks to customers in Mwanza City and other designated operational areas such as Kisesa Township. Two small water intake stations “Chakula Barafu” and “Luchelele” remain as standby intakes. Due to the topographical nature of Mwanza City, MWAUWASA operates about five main booster stations and 16 small boosters located at different points of the city for pumping water to reservoirs (water storage tanks) at elevated or hilly areas.



(a)



(b)

Figure 9: Proportion of lengths as per pipe sizes (a) and pipe materials with lengths and average age (b) for Mwanza city Water Distribution Network

As seen in the network layout Fig. 10, the trend of network expansion has increased in recent years compared to the total network length. It shows in years 2000-2009 and 2010-2019 the expansion was 29.5% and 49.1% respectively, while before 2000 the expansion was 22.4%. The topography of Mwanza City is hilly and rocky in bigger part of the city. In operations of supplying water, there are parts of the network receive inadequate water flows and pressure while other areas have 24 hrs supply. There is no fixed water supply rationing despite water production being less than water demand.

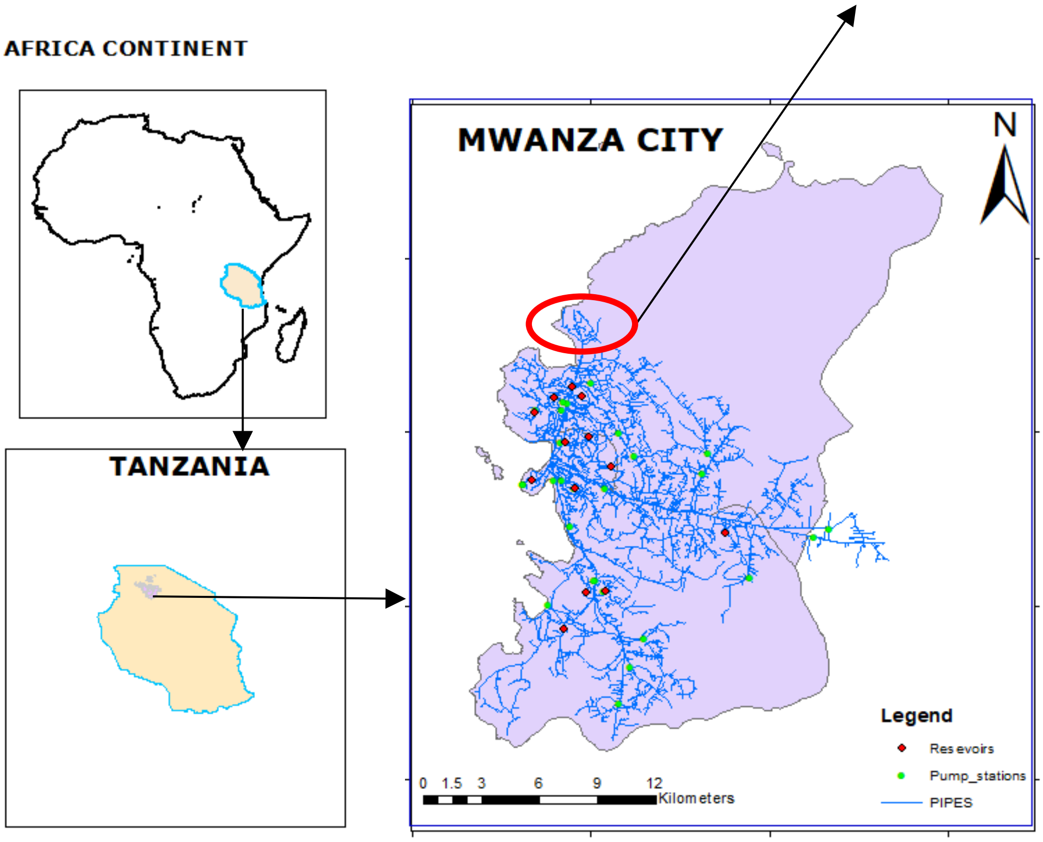


Figure 10: Map of an existing distribution network of Mwanza city and Ilemela District Metering Area

A small hydraulically isolated District Metering Areas (DMA) was established to conduct field measurements. The network has a total number of 1004 customer connections and 13 km network length. It is mainly a residential area with very few hotels and day schools; no night clubs were observed. In Fig. 11, the network layout represented by characteristics of the network shown in Table 6.

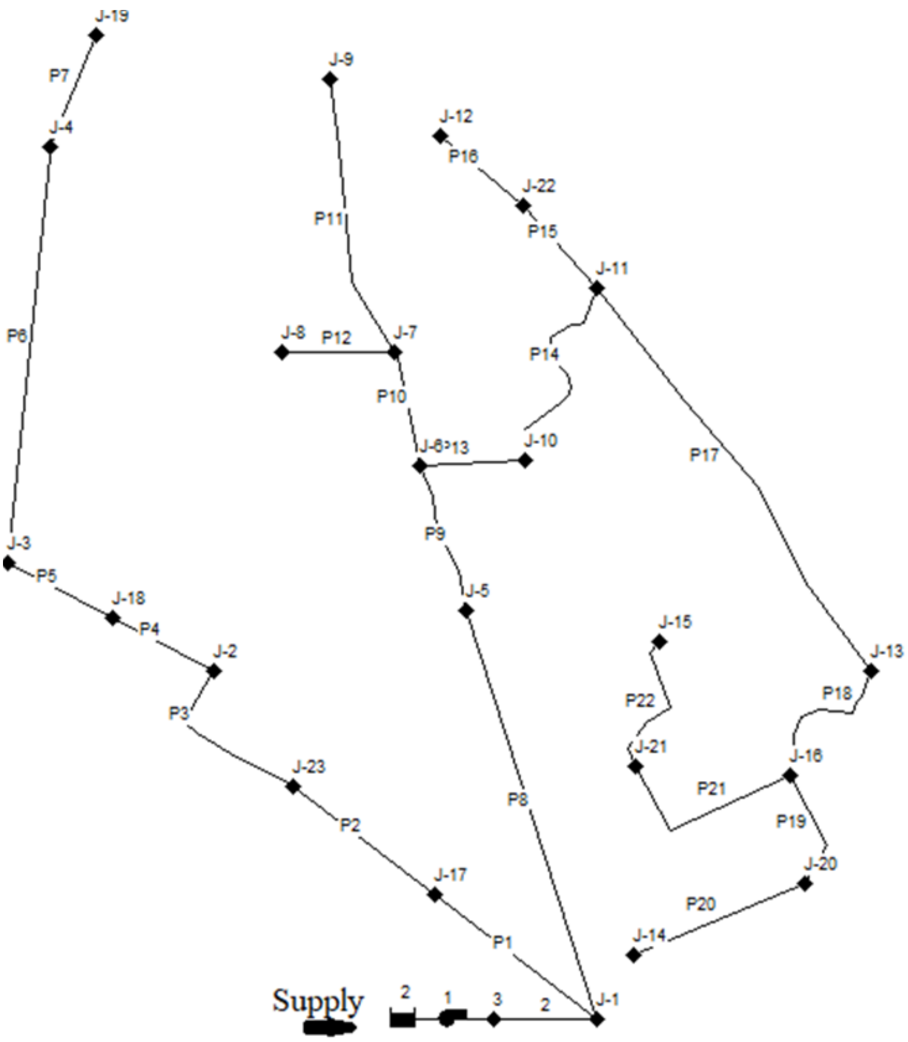


Figure 11: Layout plan of a small area Ilemela District Metering Area established for field measurements

Table 6: Characteristics of the junctions and pipes for the selected area in Mwanza city

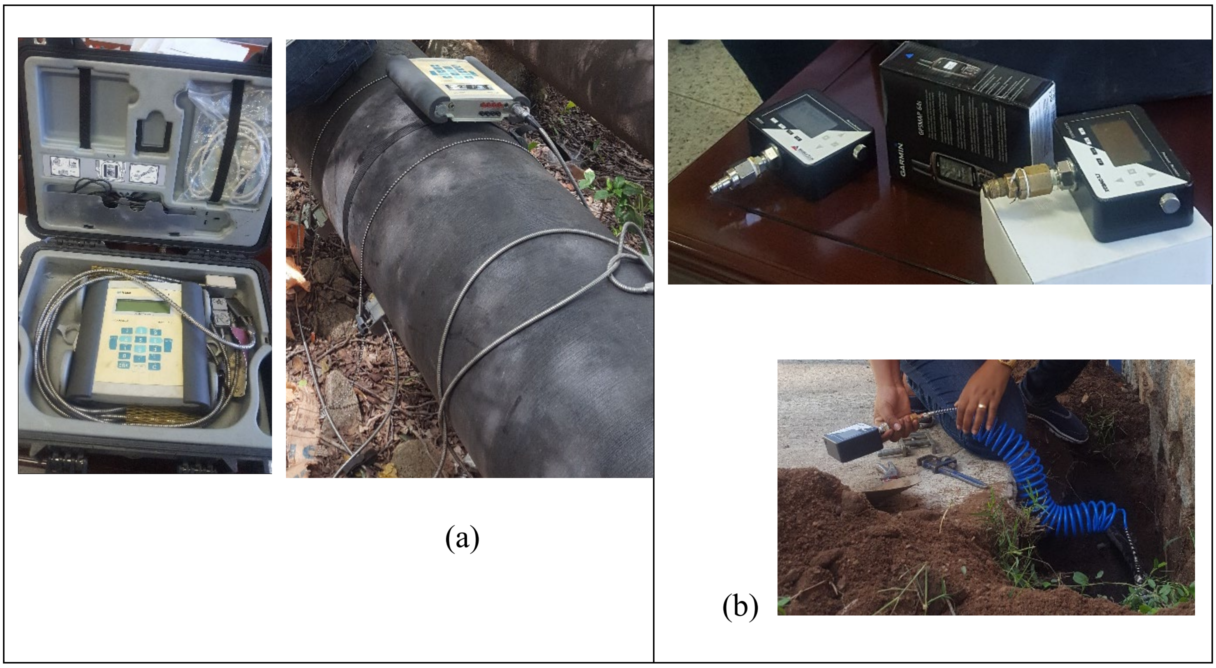
Node	Elevation	Base Demand	Link	Length	Pipe size
ID	**m**	**m^3/d**	**ID**	**m**	**mm**
J-1	1140.00	49.11	**P1**	382.0	200
J-2	1147.20	63.51	**P2**	330.8	200
J-3	1145.27	25.83	**P3**	345.8	200
J-4	1154.37	36.63	**P4**	250.9	200
J-5	1184.64	98.79	**P5**	220.3	200
J-6	1189.31	22.23	**P6**	815.6	200
J-7	1184.05	22.71	**P7**	223.2	200
J-8	1196.65	19.59	**P8**	801.1	200
J-9	1149.10	21.03	**P9**	289.3	200
J-10	1180.41	27.27	**P10**	217.3	200
J-11	1150.71	21.99	**P11**	537.4	200
J-12	1151.98	14.07	**P12**	183.4	50
J-13	1160.69	51.99	**P13**	196.1	75
J-14	1168.19	26.55	**P14**	386.3	75
J-15	1181.72	42.15	**P15**	170.5	75
J-16	1170.20	39.03	**P16**	249.1	40
J-17	1142.99	13.35	**P17**	885.1	75
J-18	1144.71	13.35	**P18**	303.1	75
J-19	1152.09	13.35	**P19**	219.3	75
J-20	1170.59	13.35	**P20**	341.8	75
J-21	1170.81	13.35	**P21**	380.6	75
J-22	1151.46	13.35	**P22**	291.2	50
J-23	1147.77	13.35			

3.2 Data collection and analysis

3.2.1 Flow and pressure measurements

Extensive field measurements were done to determine the demand pattern, base demand, nodal demand, and topological parameters of pipes and junctions. Ultrasonic flow meters for liquid model Fluxus F601 were installed at the outlets of the five big reservoirs to determine the flow rate from the tanks. The obtained hourly demand variations pattern would show the customers' consumption behaviour in a day and the hourly demand multiplier values. The average of twelve months' customer consumptions was collected from the billing database and average hourly and daily consumption computed in order to determine the junctions' base demand. Nodes or junctions were assigned the respective nodal demand based on GIS maps, satellite image and google earth guided by customers' spatial dispersal. Together with other data, including contours, pipe sizes and lengths were the inputs in EPANET 2.0 hydraulic simulation software used for modelling of the WDN.

At the inlet of the DMA, ultrasonic flow meter model Fluxus F601 was installed to record rate volume of water entering. Six locations with different characteristics of elevation and distance from the inlet were identified, and each installed the pressure loggers' (model XiLog + 2i) to record pressure in the pipes.



(a) (b)

Photos of Flowmeter (a) and Pressure logger (b)

3.2.2 Non-revenue water analysis and water balance

By using data from field measurements of flow and pressure, Minimum Night Flow (MNF) in early mornings (0300 h-0500 h) analysis was carried out to determine leakage rates. Leakages were estimated at these hours when the flow into an area is at its minimum and pressure at maximum (Cheung *et al*., 2010). The daily real loss (DRLV) was computed using Equation 12. Amount of leakage rate at nodes was calculated using Equation 13.

$$DRLV = Q_{MNF} \times \sum_{i=0}^{24} \left(\frac{P_i}{P_{MNF}}\right)^{N1} \qquad 12$$

$$Q_L = C_d x H^{N1} \qquad 13$$

where Q_{MNF} is the average minimum nightly leak flow rate (m^3/h); P_{MNF} is the average pressure in observed point for each time *t;* P_{MNF} is the average pressure between 0300 h-0500 h, $N1$ is the flow exponent depending on pipe materials whereby more flexible the pipe has higher value of N1 which was estimated at 1.5 in this study (Cheung *et al*., 2010); Q_L is the leakage flow rate, C_d is the leakage coefficient and H is the corresponding pressure which is normally considered during MNF conditions. The water balance for Ilemela DMA as seen in Table 7 was prepared by following guidelines from IWA international water balance.

Table 7: Water balance components in the Ilemela DMA

<table>
<tr><td rowspan="6">Input volume in the system, Q</td><td colspan="2">Flow rate measured by customer meters (billed), Q_m</td><td>Billed metered/unmetered consumption)</td></tr>
<tr><td rowspan="5">Water Losses/ NRW, Q_{nrw}</td><td rowspan="2">Apparent Loss (flow rate consumed but not measured), Q_{ap}</td><td>Unauthorized consumption</td></tr>
<tr><td>Customer metering inaccuracies</td></tr>
<tr><td rowspan="3">Real Losses (physical leakages), Q_{rl}</td><td>Leakage on transmission, distribution pipes, and tanks</td></tr>
<tr><td>Overflows at storage tanks</td></tr>
<tr><td>Leakage on service connections</td></tr>
</table>

3.2.3 Modelling and reliability of a water distribution network

The EPANET 2.0, which is a computer simulation model established by the United States Environmental Protection Agency was used in hydraulic simulation of the network. A WDN model was built using measured and collected secondary data. The improved looped type structure WDN was formed by adding links of specific pipe length in EPANET 2.0 software. The established model was run and analysed its correlation fitness between measured and simulated results. While conducting measurements, the operations of the entire network were monitored to record any variations to the measurements. Calibration was done as stipulated in Fig. 12 by considering values, location and time of measurements for pressure and flow data. Correlation between means of the measured and simulated values was determined to understand how best the model fits the real network.

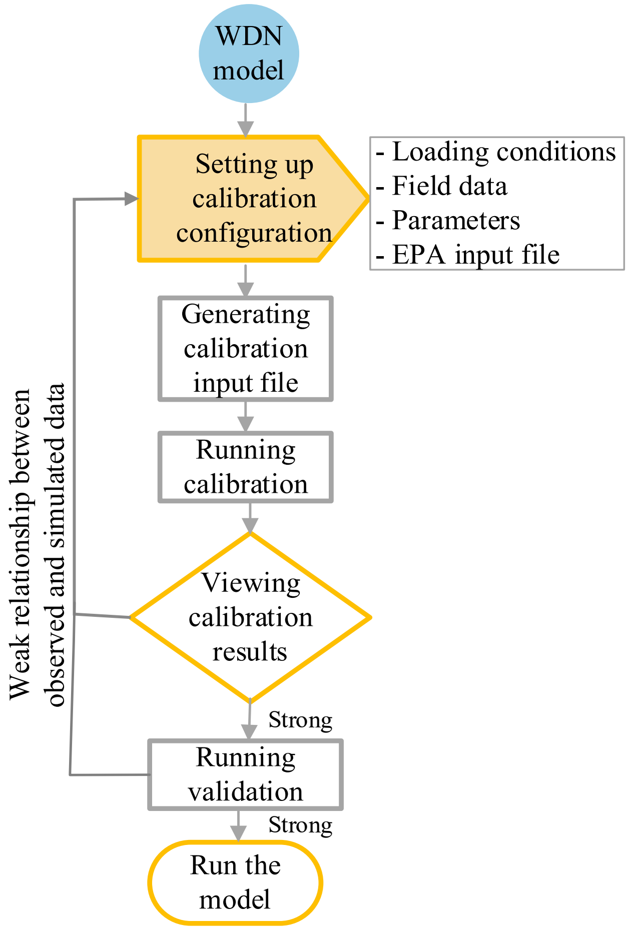


Figure 12: Calibration framework and simulation of a WDN model

The design period for the system and components were scrutinized to assess the limitations of the capacity of the system. The Hazen Williams coefficient applied was 120 considering the system was existing and consisted of uPVC and HDPE materials pipelines. The value of 5 m residual pressure at the point of use was applied as minimum head (H_{min}) beyond which there was no flow in the respective junctions. Determination of service head (H_s), which is the pressure head where the satisfaction of demand in any junction was assumed at 15 m. The assumption considered the local standards of the pressure head requirements for fire-extinction. The maximum pressure head (H_{max}) was specified at 60 m. It was used in determining the leakages and pipe bursts in the system. By using Equation 14, the satisfactory system operation was determined by checking the availability of required residual pressure at junctions (Gupta & Bhave, 1994; Shuang *et al.*, 2014). Modifications of flows and pressure were done using algorithms procedures explained in Fig. 13.

$$Q_{supplied} = \begin{bmatrix} 0\ (no\ flow) for\ H > H_{max} \\ Q_{j,req}(adequate\ flow)\ for\ H \geq H_s \\ \frac{H-H_{min}}{(H_s-H_{min})^{0.5}} * Q_{j,req}(partial\ flow) \\ 0\ (no\ flow) for\ H < H_{min} \end{bmatrix} \qquad 14$$

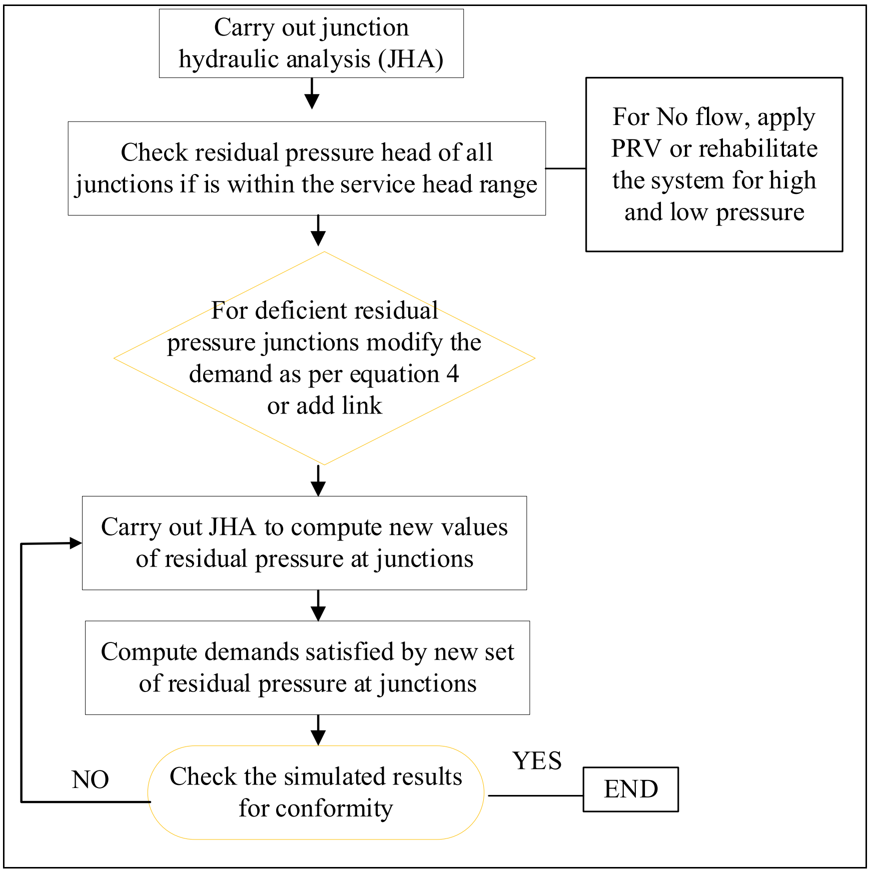


Figure 13: Flow chart of algorithms for hydraulic analysis for flow modification approach

3.2.4 Mean-time to failure and mean time to repair

The mean time to failure is generally determining the frequency of system stop from operations due to various reasons including mechanical and hydraulic failures. The average time used to fix the failure occurred is called mean time to repair. During system performance there are breakdowns occur which requires its analysis to ensure effective strategies of system improvement is attained, The idea of mechanical availability of components based on their failure records developed by Mays *et al.* (1989) was applied. A regression equation was formulated for the break rates of water pipelines using local conditions. The equations for stationary values of Mean Time to Failure (MTTF) and Mean Time to Repair (MTTR) were assumed as shown in Equation 15 and 16. The values of pipeline availability computed with the estimation that the network was ten years old. To estimate the values of K and D, the MTTR and diameters were plotted on a logarithmic scale to give more precise estimate numbers.

$$MTTF = \frac{1}{N_i * L_i} \quad 15$$

$$MTTR = K * D^a \quad 16$$

where Ni is the breaks/km/year; Li is the length of pipeline in km; D is the diameter of pipeline in mm; K is the constant; a is age of the pipes in years.

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 Flow and pressure analysis of a water distribution network

Flow and pressure are considered critical parameters of a WDN hydraulic performance, particularly when addressing the high NRW problem, so their analysis is critical and inevitable. As input parameters in this research they were main drivers of the reducing NRW and system improvement. They have direct relationship with the WDN topology and its configuration. The topology of the network principally considered configuration of WDN components, which influence the functioning of hydraulic performance of a specific system. The installed pressure loggers at six different locations within an established DMA represent the average pressure variation of various areas. The results show significant pressure variation up to 70 m between junctions, as seen in in Fig. 14.

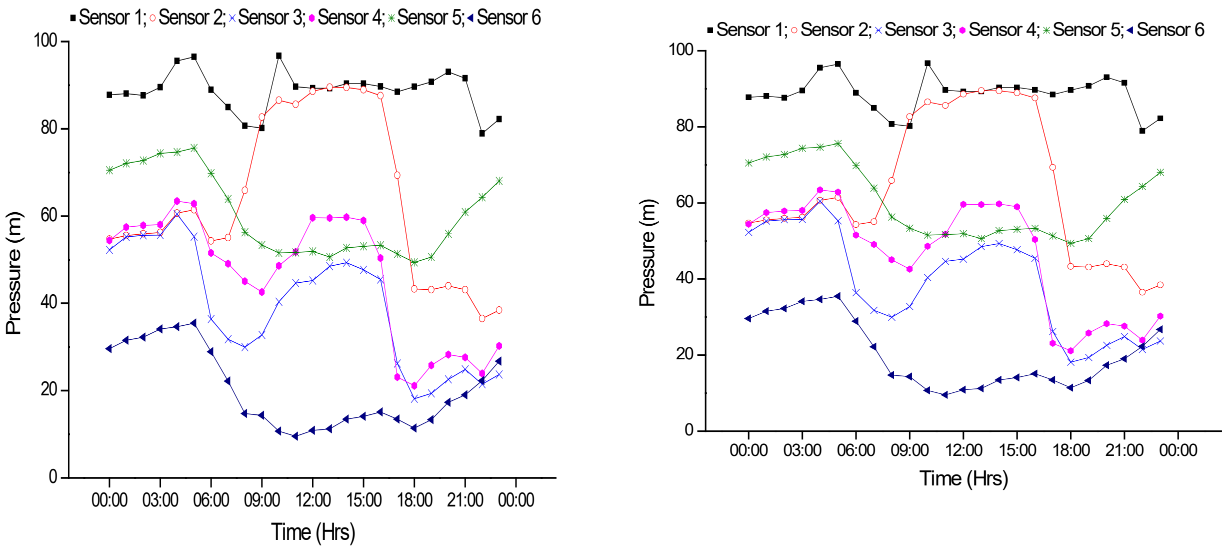


Figure 14: Average hourly pressure variation at six different locations within the study area

As one of the key parameters of the water network, it shows the hydraulic behaviour of the system, which can determine level of performance of the system. Some areas receive very high pressure (sensor 1) and some low pressure (sensor 6). In sensors 1 and 5 they receive pressure above maximum recommended (60 m) throughout the day. In sensor 2 the pressure is average except for some hours between 0600 h and 1800 h the pressure goes high. In sensor 6 there are some hours that the pressure goes down beyond 15 m. In sensors 3 and 4 the pressure is within the range of service head. Also, there are two different patterns of hourly variations in sensors 1, 5 and 6 and sensors 2, 3 and 4.

It is stipulated in the design manual of water and sanitation (URT Ministry of Water, 2009) that the pressure difference within a pressure zone should not be greater than 10 m. However, the results show a difference of about 70 m, which indicates a need to establish sub-pressure zones. It also helps in pressure management programs to avoid excess pressure, hence will save energy and prolong water infrastructure lifespan. The high energy loss within the supplying system is observed as the pressure head entering the system is significantly bigger than the residual pressure in some areas. More investigations could provide more information on the high amount of head-loss if leakages cause it. There are observed different behaviours of water consumption within the area. During high consumptions, the pressure tends to show dropping trend and vice versa. The patterns of hourly pressure variations may help in the setting of pressure management tools as well as water rationing. However, it needs more studies to understand the exact reasons for variations.

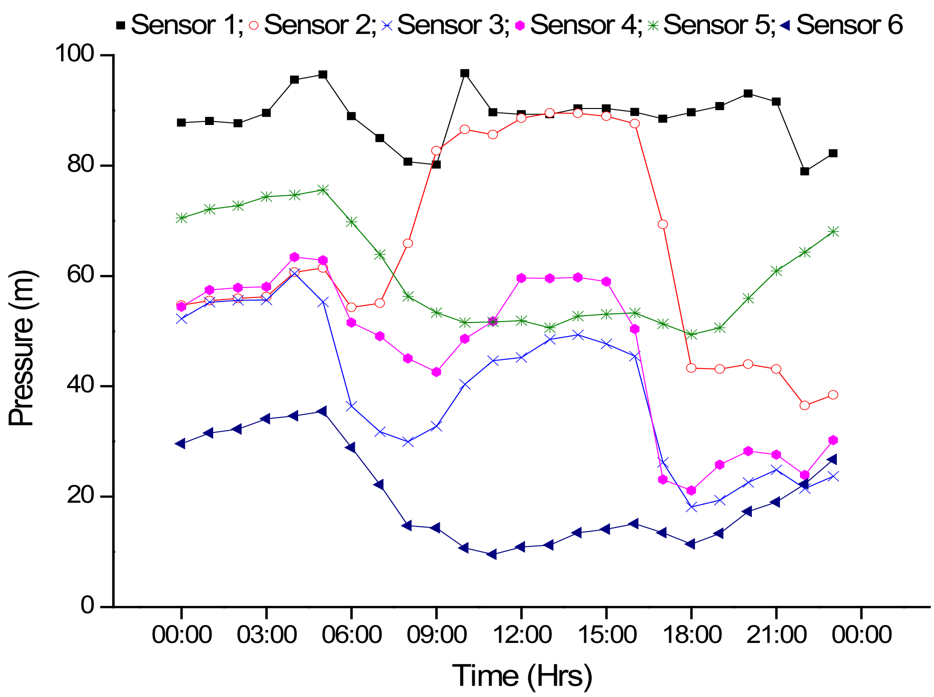


Figure 15: Average hourly pressure variation at six different locations within the study area

The second key parameter in hydraulic performance of a water network is flow rate. The amount of water entering the system should be enough to meet the demand of a specific area. The data of flows recorded by the flow meter installed at the inlet of a DMA are shown in Fig. 16. The trend of water consumption for 12 days shows similar flow and pressure patterns with two peak flows. The average flows during early morning hours between 0300 h – 0500 h ranges from 30 to 50 m^3/h while pressure goes up to 100 m. For three consecutive days from 19^{th} to 21^{st}, both morning and evening peak hours were small compared to other days. Also, on 17^{th} the early morning hours show low flows compared to other days.

The daily fluctuations of inflows to the DMA indicate that some days, the system may receive less water. It then requires monitoring and more investigations on feedback of water availability from customers to find out its relationship with water inflows. This can help set up the limits of adequate water entering the system. Long-time monitoring of flow patterns is recommended to see causes of variations whether the amount of water in the entire system has reduced or change consumptions patterns. The information could be included in water supply operations and rationing. The flow spikes observed give signs of significant flow fluctuations that need to be monitored to avoid damage of the infrastructure. High values of early morning hours flow indicate high leakage levels within the area. Assessment and monitoring of background and burst types of leakage should be done. Principally, these flow values are combinations of leaking water from pipes and legitimate customer night use. However, the volume of low flows mentioned can be inclusive of legitimate flows, though in this study it is negligible. During early morning hours in residential areas, no water use is expected except for toilet flushing which is insignificant. In the case where there is substantial night use like night clubs or other night activities, it is advisable to install flow meters to quantify the legitimate use. The general trend shows patterns of flow and pressure at the inlet point go in the opposite trend; during low consumptions, the system experiences high pressure.

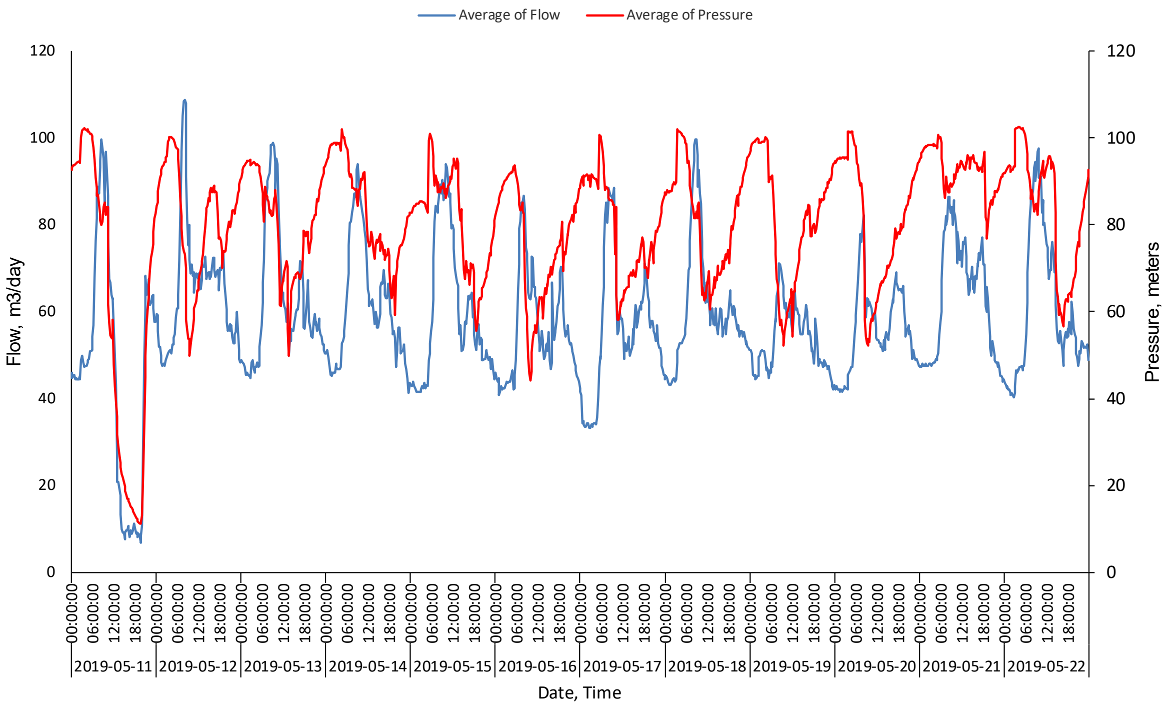


Figure 16: Flow rate entering the system showing peak and minimum flows

4.2 Water balance

There are four components of water uses: Billed authorised, unbilled authorised, apparent losses, and real losses. The results computed from the MNF analysis gave the value for the daily real loss volume (leakage). Other components of NRW as described by the IWA international water balance were estimated as presented in Fig. 17. The amount of water consumed shows 50% of water supplied to the system, and the remaining percentage is water loss. The percentage of computed leakage indicated about 87% of water loss originates from leakages, and the remaining percentage comes from apparent loss. No unbilled authorised consumptions within the study area. The computed 50% NRW in a DMA is in the higher side compared to the national benchmark of 20%. It is uneconomical to loose half of the produced water that is treated and energy used for its distribution. This shows why it is important to carry out zone by zone analysis to understand their contribution to the city's overall NRW. The higher contributor to the overall NRW should be prioritized in NRW reduction activities. In this case, as leakages found significantly high, it should be given priority in strategies for controlling. It should also attract high investment to curb the problem of NRW. The water audit shows a localized water balance that indicates NRW equivalent to the water loss.

From the results, there was about 50% of NRW in the study area, whereby 87% of these were caused by leakages, and the quantity of leakage was 44% of the total supplied water. Since 50%

NRW is higher than Mwanza average of 37%, it is important to carry out zone by zone analysis to understand their contribution to the city's overall NRW. The zonal values can be combined and used to develop appropriate NRW reduction strategies in the city, which will reduce operating costs by saving the lost water.

In the IWA international standard water balance, there are four main components of water uses in a WDN i.e. billed authorised, unbilled authorised, apparent losses, and real losses. In the present study, there was no unbilled authorised consumption. After the water audit was conducted, a customized water balance was developed which regarded NRW equivalent to the water losses. Comparison of components of the NRW study area and the entire WDN gave a summary of leakage (real loss), apparent loss, and billed values in comparison to the total water supplied and NRW for the selected area and the entire city. The summary is shown in Table 26.

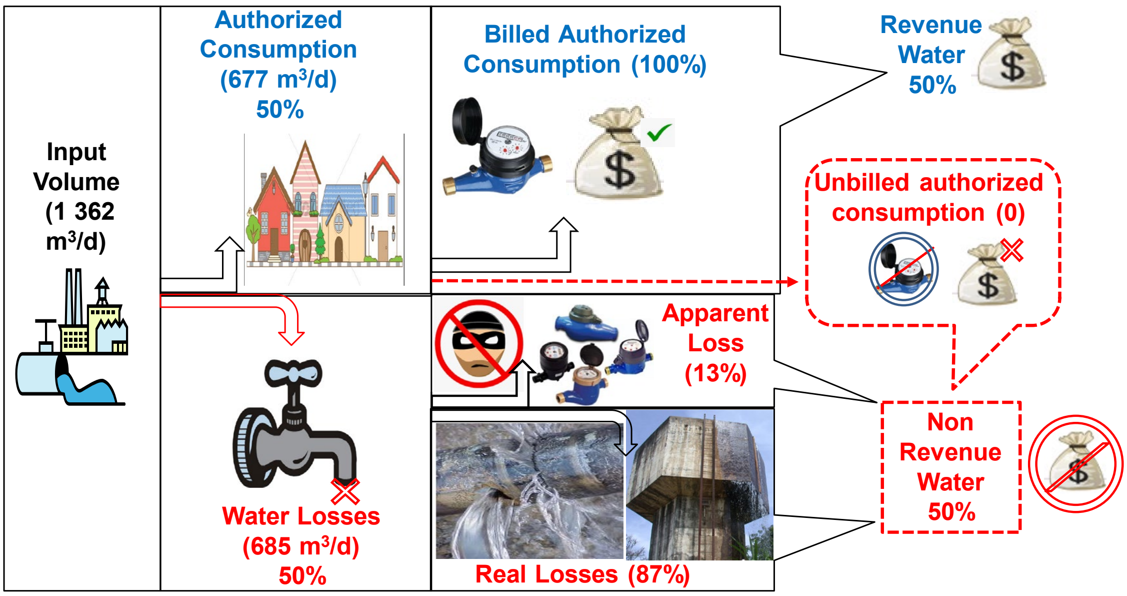


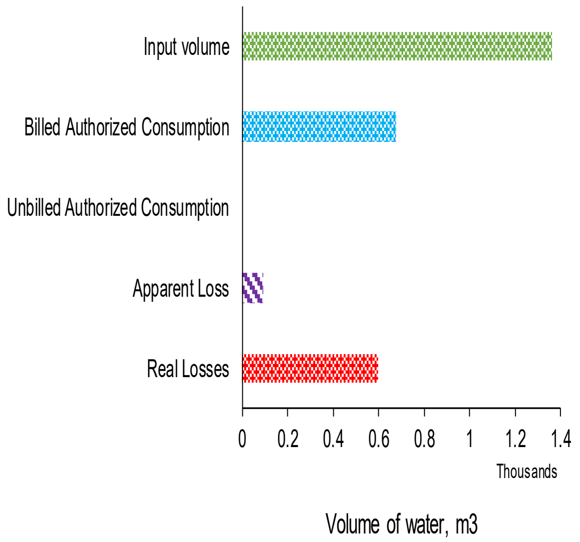
Figure 17: Pictorial presentation of the water balance of the analysed District Metering Area

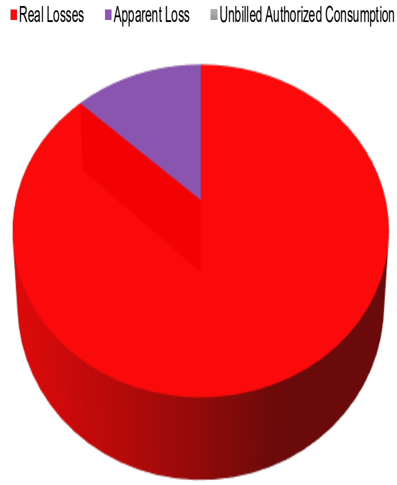
The comparison of different water uses in DMA and the entire network gave details of leakages (real loss), apparent loss and billed values in relation to total water supplied and NRW as shown in Table 8. The water lost through leakages is about 44% of the total supply which is higher compared to about 6% apparent loss. The percentage of billed water for the entire city is 63% which is higher compared to 50% in a DMA. Comparison of components of NRW study area and the entire WDN shows a summary of leakage (real loss), apparent loss and billed values in comparison to the total water supplied and NRW for the selected area and the entire city.

Table 8: A detailed water balance of the DMA in comparison to the entire city

Study area	$Q_m / Q = 0.497$	
	$Q_{ap} / Q = 0.066$	$Q_{ap} / Q_{nrw} = 0.131$
	$Q_{rl} / Q = 0.437$	$Q_{rl} / Q_{nrw} = 0.869$
Mwanza city	$Q_m / Q = 0.630$	

Where Q is the system input volume; Q_m is the flow measured by customer meters (billed); Q_{nrw} is the NRW; Q_{rl} is the real loss/leakage; Q_{ap} is the apparent loss (flow rate consumed but not measured). Elaboration of components of water balance indicated in Fig. 18 a, b and c shows the magnitude differences of each component relatively.





Water balance chart (a)

NRW chart (b)

<table>
<tr><td rowspan="9">System Input Volume 100%</td><td rowspan="4">Authorized Consumption 50%</td><td rowspan="2">Billed Auth. Cons. 100%</td><td>Billed metered cons.</td><td rowspan="2">Revenue Water 50%</td></tr>
<tr><td>Billed unmetered cons.</td></tr>
<tr><td rowspan="2">Unbilled Auth. Cons. 0</td><td>Unbilled metered cons.</td><td rowspan="7">Non-Revenue Water (NRW) 50%</td></tr>
<tr><td>Unbilled unmetered cons.</td></tr>
<tr><td rowspan="5">Water Losses 50%</td><td rowspan="2">Apparent Loss 13%</td><td>Unauthorized con.</td></tr>
<tr><td>Customer metering inaccuracies and data mishandling</td></tr>
<tr><td rowspan="3">Real Losses 87%</td><td>Leakage on pipelines</td></tr>
<tr><td>Leakage & overflows at tanks</td></tr>
<tr><td>Leakage on service connections</td></tr>
</table>

IWA water balance (c)

Figure 18: Water balance for the study area

4.3 System reliability

It is important to assess reliability of the system before proceeding to hydraulic analysis and modelling. Satisfactory pressure and mechanical connectivity types of reliability were evaluated. The reliability based on the number of satisfactory times the system has the required pressure, R_s was 52%, and for connectivity to the source $R_{s,c}$ was 76.6% (Table 9). The first type is principally depending on the level of residual pressure in the system, and the later is based on the system sustainability due to age, length, pipe size. The MTTR is higher for bigger pipes, and it reduced as pipe sizes decreased. It has similar trend for MTTF while opposite trend observed for break rates. Its equation was formulated using break rate regression of the log-log plotting of time to failure and diameter of pipes, as shown in Fig. 19. The values of number of breaks estimated based on local conditions, field experience, and utility records. The MTTF is mostly affected by the length of pipes; it increases as the length is reduced. It drives the reliability output; when increases the reliability also increases.

Both types show the reliability of above 50%, which is reasonably good. The results indicate a high probability of failure of the system due to presence of pressure that is out of the required range of 15 m – 60 m. About 48% of the junctions observed to have pressure outside the range. When pressure is too high may cause pipe burst and lead to valves closure to restrict water flowing. Also, low pressure may not have enough energy to reach part of the network and may cause complaints from customers. These decrease the level of the service as customers in several parts of network will have no water. Pressure should always be maintained to an allowable range to increase system reliability. For mechanical connectivity, it shows 6714 hours out of 8760 hours in a year the system is connected to the source. During that time the consumers will be able to receive water while the remaining hours the system will be out of the service. During system life span, the infrastructure may deteriorate due to age, which increases number of pipe breaks and consequently number of times for repair works. Thus, reliability based on average time the system fails, and time spent to repair decreases. Changes in network configuration layout to looped structure would increase the availability of the connectivity to the source. The loops interlinks provide alternative routes of supplying when the system is out due to failure and repairs. Also, the looped system has high connectivity reliability (Kansal *et al.*, 1995; Gheisi *et al.*, 2016), small chances of hydraulic failure; only a small isolated part is affected during repair works, and in fire-fighting, more water may be re-directed to the needy part.

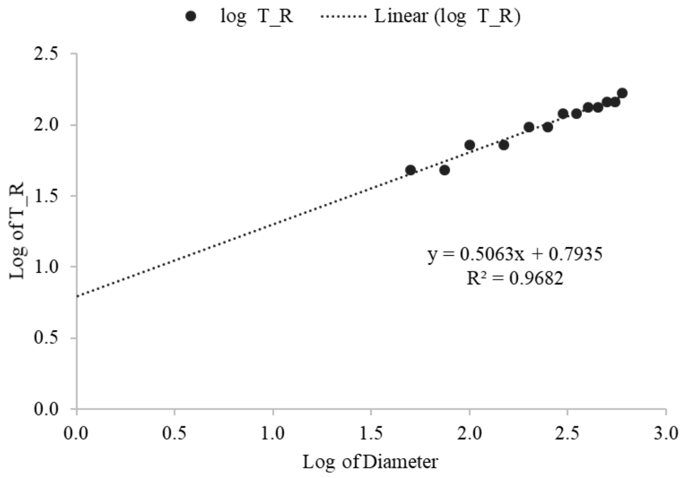


Figure 19: The mean time to repair

Linear model Polynomial:

Log of MTTR = a*Log Dia. + Log K

Function f(x) = m*x + c

y = 0.5063x + 0.7935 and goodness of fit R-square is 0.9682

Log MTTR = y; Log Dia. =x; a= slope =0.5063; Log K = y- intercept = 0.7935

The equations are: $MTTR, x_i = 6.3 * D^{0.5}$ (in hours) and $MTTF, y_i = \frac{1}{N_i * L_i}$ (in years)

where x_i is the Mean Time To Failure (MTTF) of i^{th} pipe in years; y_i is the Mean Time To Repair (MTTR) of i^{th} pipe in years; N_i is the breaks/km/year; t is the pipe age in years, D is the pipe diameter in mm; L_i is the length of the pipe in km; p_i is the pipeline availability.

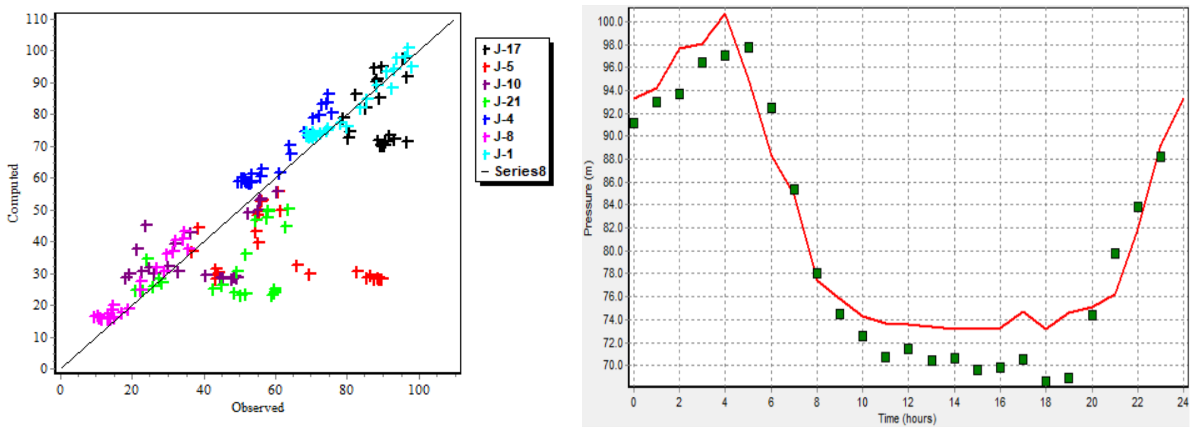
Table 9: The pipeline availability based on mechanical reliability

ID	Length m	Dia mm	MTTR (y_i) YRS (x10^-3)	Ni br/km/yr	MTTF (x_i) (YRS)	p_i $x_i/(x_i+y_i)$
Pipe P1	382.04	200	10.1707	3.5	0.7479	0.9866
Pipe P2	330.80	200	10.1707	3.5	0.8637	0.9884
Pipe P3	345.82	200	10.1707	3.5	0.8262	0.9878
Pipe P4	250.86	200	10.1707	3.5	1.1389	0.9911
Pipe P5	220.31	200	10.1707	3.5	1.2969	0.9922
Pipe P6	815.59	200	10.1707	3.5	0.3503	0.9718
Pipe P7	223.21	200	10.1707	3.5	1.2800	0.9921
Pipe P8	801.09	200	10.1707	3.5	0.3567	0.9723
Pipe P9	289.3	200	10.1707	3.5	0.9876	0.9898
Pipe P10	217.25	200	10.1707	3.5	1.3151	0.9923
Pipe P11	537.35	200	10.1707	3.5	0.5317	0.9812
Pipe P12	183.44	50	5.0854	6	0.9086	0.9944
Pipe P13	196.14	75	6.2283	5	1.0197	0.9939
Pipe P14	386.31	75	6.2283	5	0.5177	0.9881
Pipe P15	170.53	75	6.2283	5	1.1728	0.9947
Pipe P16	249.12	40	4.5485	6	0.6690	0.9932
Pipe P17	885.12	75	6.2283	5	0.2260	0.9732
Pipe P18	303.06	75	6.2283	5	0.6599	0.9907
Pipe P19	219.32	75	6.2283	5	0.9119	0.9932
Pipe P20	341.84	75	6.2283	5	0.5851	0.9895
Pipe P21	380.57	75	6.2283	5	0.5255	0.9883
Pipe P22	291.23	50	5.0854	6	0.5723	0.9912
						0.7664
						76.64%

4.4 Model calibration and simulation

Calibration is the critical activity in modelling as it adjusts the model to reduce errors in its fitness to the real system. It compares the observed data taken during field measurements and simulated results. In Fig. 20 a, the determined correlation between means of pressure heads for the 7 junctions was 0.87. Junctions J-1, J-4, J-8, J-10, and J-17 were observed good fitness compared to junctions J-5, and J-21. In Fig. 20 b, at a specific junction, one pressure fitness indicates the strong relationship between the two values. The results in Fig. 21 shows the 24 hours flow variations for 12 consecutive days fluctuating due to the uncertainty of the real WDN.

Generally, the correlation obtained indicates the strong relationship between the two values. However, some junctions did not fit well, so it required model adjustment. Several measurements in different seasons are proposed that can be applied in model adjustments to reduce fitness errors. Assessment of the spatial distribution of the measured junctions does not show any trend concerning model fitness.



(a) (b)

Figure 20: Calibration results of the two measured and computed pressure for the seven different points and J-1 respectively

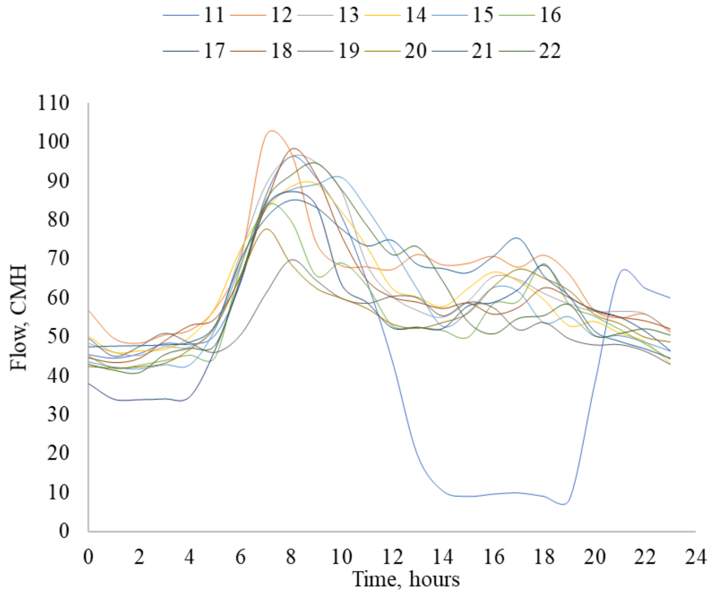


Figure 21: Monitoring of water flow for 12 days showing stochasticity of the Water Distribution Network

The model was adjusted using the four calibration variables: Leak detection, pipe status, demand factor and roughness (Hazen William-HWC). Each was run under two optimization scenarios, i.e. early morning hours and 24 hours (all times), as shown in Table 10. In scenario 1 the leakage detection variable has the lowest fitness while combined has the highest. Scenario 2 run, gave almost the same trend except for slight differences observed in demand adjustment and roughness-constant variables. The first scenario has lower fitness error values compared to the second scenario.

On low demand hours, which occurs in the early morning between 0000 to 0500 h, the leakage detection performed best compared to the rest variables. In real conditions, these during these hours there is almost no consumptions; that is the reason the model fits well. Also, the first scenario seems to perform better than the second one. The calibration process is applying a Genetic Algorithm (GA) principles for model fitness. When the range of hours in adjustment processes is extensively big, the GA search method shows a reduced GA efficiency. High values of summation of fitness errors imply less efficient of the GA. In order to obtain the best fit and viable solution, it requires a proper judgement of conditions/situations occur in real network to make decisions in applications of scenarios and variables. In pipe status variable, closed status observed implying no flows. Though the existing network is a branched style, so any closure status could cause failure to get water at that particular part of network. However, this is not the condition at field; it needs investigations.

Table 10: The summation of errors for different calibration variables under two scenarios

Variables > Scenarios 1and 2 v	**Leakage**	**Demand**	**Roughness**	**Status**	**Combined**
Early morning	0.132	0.174	0.178	0.180	0.852
All times	1.460	1.555	1.535	1.538	3.495

The leakage detection simulation between 0000-0500 h spotted six critical junctions that require attention in monitoring and controlling of operations (Fig. 22). The emitter coefficients with color and sizes coded indicate 0.01, 0.04, 0.07, 0.31, 0.31 and 0.35 in junctions J20, J18, J1, J5, J8 and J4 respectively. The bigger sized node in J4 has high leakage rate compared to the rest and J20 has lowest.

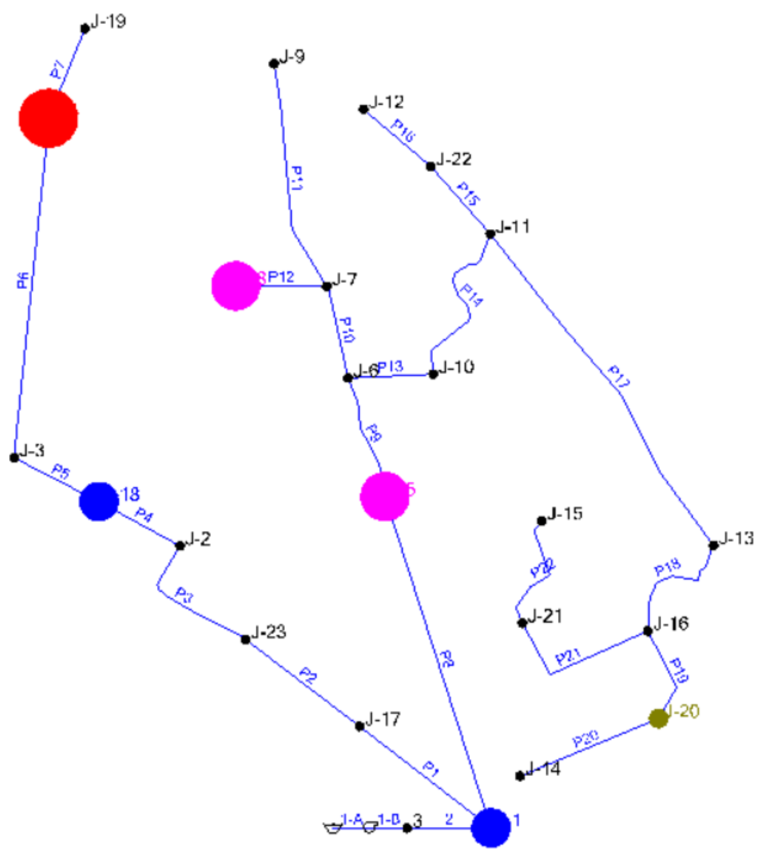


Figure 22: Leakage detection nodes showing 6 leak points varied by size of circles proportional to amount of leakage flow at junctions within a sampled District Metering Areas

4.5 Pressure propagation at junctions of Water Distribution Network

Simulation results observed in Fig. 23 a, b, c and d show the propagation of average pressure in each junction for 24 hours. In (a) between 0000-0500 h, the pressure heads are above 35 m for all junctions while in (b), (c) and (d) pressure heads drop to 15 m. For (b) and (c), maximum pressure heads do not go above 80 m while in (a) and (d) they approach 100 m. The observed hourly fluctuation is high in all junctions throughout a day. Junctions 1-3, 9, 17-19 and 23 showed the highest pressure while junctions 8, 15 and 21 had lowest residual pressure.

As seen in measurement data, the pressure in the early morning is high because the demand is minimal and the pressure in the system is maximum. Also, in night hours the system experiencing slightly high pressure compared to day time. During the day time, there are many activities, so the demand is high, which causes pressure reduction in the system. Pressure fluctuations observed to be too high for some junctions compared to other junctions. It needs more attention and considerations of critical junctions during operations, improvements of the system and expansion of the network (Adedeji *et al.*, 2017).

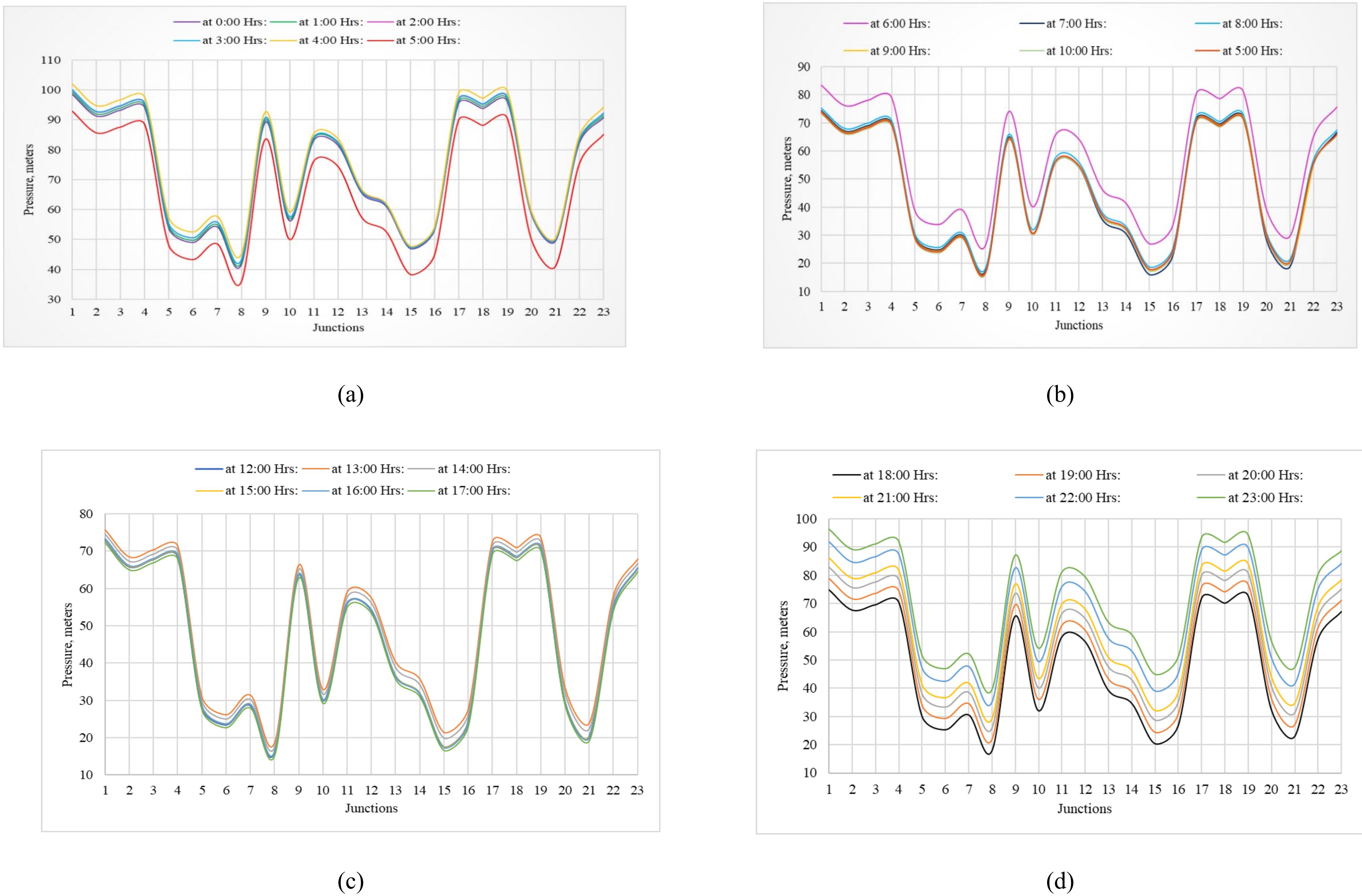
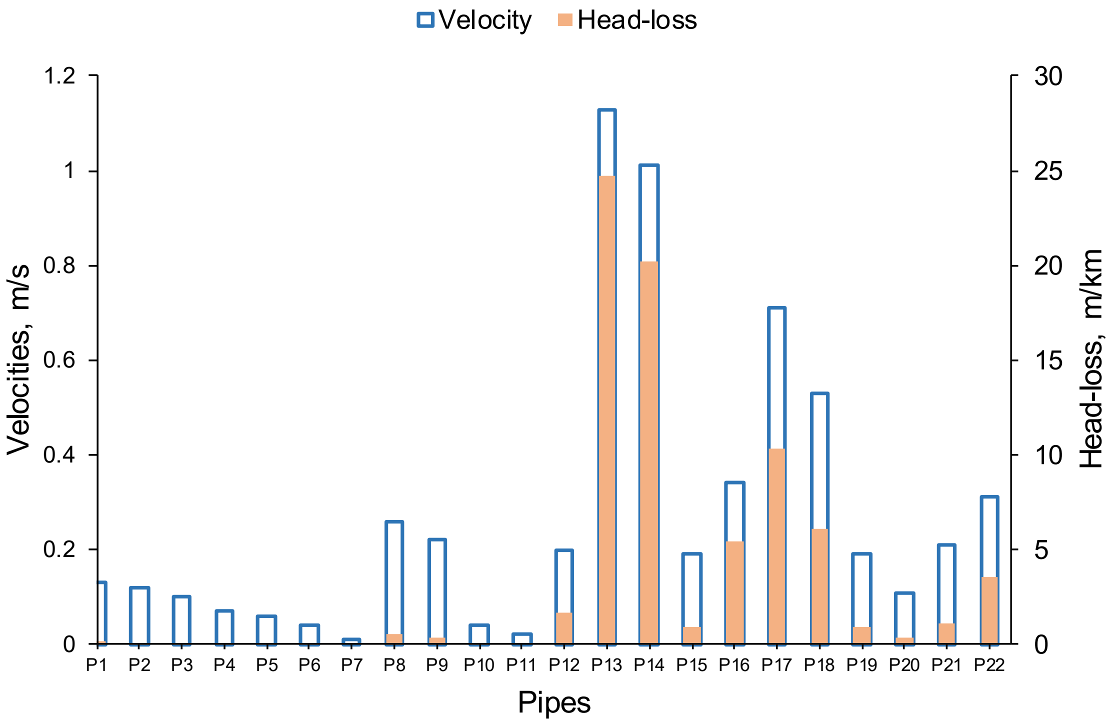


Figure 23: Pressure propagation for 23 junctions for 24 hours of a day

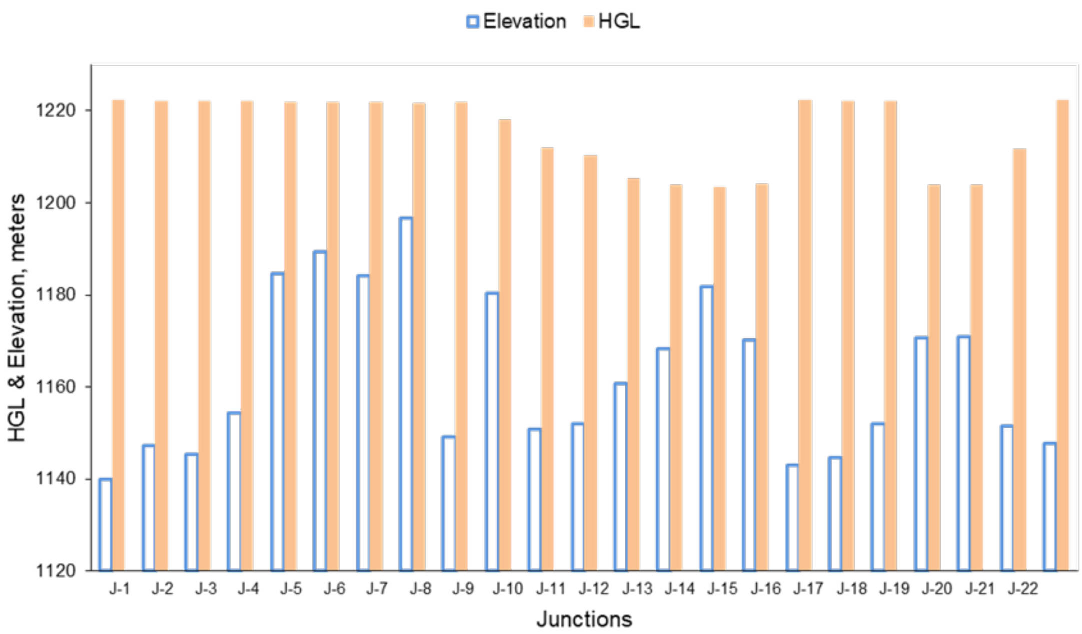
Simulated results of velocities, head-loss and Hydraulic Grade Line (HGL) shown in Fig. 25 a, b indicate accepted velocities in 3 (>0.6 m/s) out of 22 pipes. The same three pipes have the highest head-loss values. The trend to most pipes shows an increase of head-loss as velocities increase except for Pipes P20, 15, 12, 9 and 8. Pipes 13, 14 and 17 Pipe sizes looked bigger than the present water demand of the area. They store water in the network like reservoirs, which may create water stagnation and increase system pressure. Long small service lines connected to big pipes perform like 'orifices' discharging water to customers. The small pipes are at risk of bursting due to high pressure that may reduce life span of the infrastructure. The stagnant water may deteriorate quality of water, so further studies on its assessment are critical. The over-design of many systems in the initial years of their construction is one of the challenges in water projects. For example, this case study for more than ten years, the system is working far below the design capacity. Further studies in selecting the design approach whether flexible or stage designs are practical in the situation of stochastic demand growth in fast-town expanded. There are limited existing secondary distribution lines in the area that led to formation of a spaghetti-type of service lines as seen in Fig. 24. The simulation indicates that changing WDN topology by shifting supply direction through addition of secondary pipes is important to reduce the 'orifice-like' operation of the service lines. The service lines have low capacity to withstand high flows and pressure. This will reduce leakages in smaller and long "spaghetti" lines. The additional secondary pipelines create loops that improve system air circulation and reducing water stagnation.

For the HGL in Fig. 25 (b), it is shown about 12 junctions out of 23 experience residual pressure above 60 m (allowable for the distribution network). These junctions are susceptible to high leakage rates due to pipe bursts; they should be monitored cautiously during operations. Mostly, these junctions found on a right-hand side branch of the network (along lakeshores). The population density of this part of the network is lower compart to the other areas (refer to Fig. 10). However, big pipe sizes mostly 200 mm found in the area irrelatively to the existing current demand. It emphasizes integrating uncertainty of town growth in designing of economical WDN and stage or flexible designs.

Figure 24: Picture showing spaghetti service lines in the study area extended for long distance from the mainline



(a)



(b)

Figure 25: The simulated results of pipes and junctions of the selected network

4.6 Junctions hydraulics of a water distribution network

Junctions were assessed to ensure each one receives adequate pressure (within the range) for system improvements, increase level of services efficiency and leakage reduction. High leakage levels factually influenced by high pressure within the WDN (Al-washali *et al.*, 2016). The Break Pressure Tank (BPT) simulated in the existing system and indicated a good performance by reducing pressure head to an allowable range of 20-30 m. However, it showed a system failure when simulating future demand growth for a few years (less than five years). This scenario cannot perform at the required pressure for the whole operating period before the end of the design horizon. Though this alternative is cheap in operation and most offered in Tanzania compared to pressure reducing valve (PRVs), it has limitations in regulating the pressure of the system. The latter is too mechanised and needs regular maintenance and repairs, while BPT mainly requires space and initial construction costs.

The pressure reducing valve (PRV) was the next simulated alternative for the objective function of leakage minimization. The PRV optimized in quantities, setting and to location as decision variables with the constraint minimum pressure of 20 m to continue providing enough pressure (Table 11). Junctions J-9, J11, J12 and J22, currently supplied from the right-side branch, has high pressure above 60 m, and it reduced when supply changed to left-side branch. High values were also obtained when one PRV was set for both branches, with the pressure reaching 75 m. By applying leakage formula and emitter coefficients values obtained earlier set-up 1, 2 and 4 resulted in minimal leakage rate of 95%, 81%, and 93% of the current leakage, respectively.

Table 11: The simulated pressure values in m for the four set-ups of PRVs

Node	Before installing PRV	Installing one PRV at the entrance (1)	Installing two PRVs for each branch (2)	Installing two PRV and change water flow direction for some links (3)	Installing two PRV and loop the entire network (4)
J-1	82	80	37	37	79
J-2	75	73	29	29	72
J-3	77	75	31	31	74
J-4	68	66	22	22	65
J-5	37	35	35	35	35
J-6	33	31	30	30	30
J-7	38	36	35	35	35
J-8	25	23	23	23	22
J-9	73	71	70	27	70
J-10	38	38	38	38	38
J-11	61	66	65	66	67
J-12	58	65	64	25	67
J-13	45	53	52	53	54
J-14	36	45	44	45	46
J-15	22	31	30	31	32
J-16	34	43	42	43	44
J-17	79	77	34	34	76
J-18	77	75	32	32	74
J-19	70	68	25	24	67
J-20	33	43	42	43	43
J-21	33	42	42	42	43
J-22	60	65	65	25	66
J-23	74	72	29	29	71

(1) PRV set at 80m, (2) at RHS (60m) & LHS (35m); (3) RHS (40m) & LHS (30m); and (4) RHS (60m)

The set-up 1 and 4 gave more or less similar results, while set-up 2 shows efficiency improvement. Set-up 3 indicated appropriate pressure in the entire network with which is the best set-up in leakage reduction. The best scenario of PRV optimization observed in set-up 3 of two PRVs each installed in the existing two branches. The simulated pressure indicates that leakage will be 46%. Pressure reduction has shown a direct impact on leakage minimization. It is one of the four pillars of physical loss reduction, which including also: active leakage control, pipeline, and assent management (selection, installation, maintenance, rehabilitation, replacement) and speed and quality of repairs.

A pipe joining the four junctions mentioned above was added to make a looped network and changed their supply direction from low to high-pressure branch. These junctions showed best results when shifted to the high-pressure zone. This reveals that one DMA (pressure zone) required two pressure sub-zones. It emphasizes the idea of sub-zone establishment for areas of high-pressure variations. The location, number, and setting of pressure reducing valves to optimal feasible performance in terms of pressure limits should be frequently assessed and fixed as per the demand growing.

4.7 Present and future system capacity torelance

47.1 The results of the hydraulic simulation of different demand patterns are provided in Table 12

At the time of a study, the estimated system average demand: 676 m^3/d, inflow: 1362 m^3/d, and age of 10 years old. According to prevailing local conditions and historical data, the demand was approximated at 1136 m^3/d and 1596 m^3/d after 10 and 20 years, respectively. It is possible that when the demand goes higher, some of the junctions will have residual pressure head in no or partial flow modes. For example, junction 15 will have partial flow with a residual pressure of 13.3 m. Under scenario 110 yrs, the addition of a pipe link revealed that all the junctions would receive water in full mode, including junction 15 with a residual pressure of 40 m. By applying the partial flow Equation (14) described earlier, demand was changed and then simulated to get new values of junction demand and pressure. Demand series of 20 years indicated that scenario 120 yrs would continue to allow water withdrawal from all junctions in full mode. However, scenario 220 yrs showed that some junctions like 14, 15, 16, 20 and 21 would be in partial and no flow modes which need change of flow and pressure at junctions as per partial flow Equation. Scenario 220 yrs will have fewer leakages though it requires a regular revisit of junctions as residual pressures observed to drop beyond requirements. The model simulation results indicated that, after some years, part of the system will receive unreliable pressure (less than 5 m) if the current trend of random network expansion and demand growth continues.

Table 12: Simulated pressure values after modifications of flow by the additional supply of reducing demand forecasting

Jun. ID	Elev. m	Current	10yrs demand, do nothing	10yrs demand, add supply	10yrs demand, reduce demand	20yrs demand add supply	20yrs demand reduce demand
J-1	1140	84.5	82.6	82.6	82.6	81.8	81.8
J-2	1147	77.3	75.3	75.3	75.3	74.4	74.4
J-3	1145	79.2	77.2	77.2	77.2	76.4	76.4
J-4	1154	70.1	68.1	68.1	68.1	67.2	67.2
J-5	1185	39.7	37.6	37.6	37.6	36.4	36.4
J-6	1189	35.0	32.8	32.8	32.8	31.6	31.6
J-7	1184	40.3	38.0	38.1	38.0	36.9	36.8
J-8	1197	27.6	25.1	25.2	25.1	23.6	23.5
J-9	1149	75.2	73.0	73.0	73.0	71.8	71.8
J-10	1180	42.1	36.8	39.9	37.0	37.0	31.9
J-11	1151	69.0	58.6	67.2	59.3	61.8	47.9
J-12	1152	67.5	56.2	64.8	56.9	57.9	44.0
J-13	1161	54.8	38.3	55.5	39.7	48.9	21.5
J-14	1168	46.4	28.3	47.6	29.8	40.5	9.9
J-15	1182	32.1	13.3	40.0	15.4	38.5	-4.4
J-16	1170	44.5	26.6	45.9	28.2	39.2	8.6
J-17	1143	81.5	79.6	79.6	79.6	78.7	78.7
J-18	1145	79.8	77.8	77.8	77.8	76.9	76.9
J-19	1152	72.4	70.4	70.4	70.4	69.5	69.5
J-20	1171	44.0	26.0	45.3	27.6	38.3	7.7
J-21	1171	43.6	25.4	45.8	27.1	39.5	7.3
J-22	1151	68.3	57.8	66.3	58.4	60.8	46.9
J-23	1148	76.7	74.8	74.8	74.8	73.9	73.9

Current demand - 676 m^3/d; 10yrs demand - 1136 m^3/d; 20yrs demand -1596 m^3/d Elv.=elevation; Dem.=demand ; Press.=pressure; CDM=cubic meters per day;

CHAPTER FIVE

CONCLUSION AND RECOMMENDATIONS

5.1 Conclusions

Despite many efforts of reducing NRW, the same has persisted in high levels above the recommended limits and standards to most water utilities. Uneven pressure distribution found in the system created complaints of no or partial flows from water consumers located in high elevation areas. Numerical analysis and model simulation methods determined the appropriate approach which fitted for the unplanned network and fast-growing conditions. Measurement and monitoring of the two key parameters: flow and pressure head in the WDN were the major variables showed substantial changes in system performance. Their optimization showed improved operations, hence high system reliability and service satisfaction. Another finding in this study was that, the existing configuration of the WDN contributed part of the entire NRW. The operations of unplanned network not integrated with the stochastic town expansion were found as a driver of the high water loss through leakages. The less-dense areas were found with large pipe systems resulted in low flows due to low velocities hence high pressure, which invited more potential leakages. The low velocities observed caused problems of poor water quality due to water stagnation. High pressure increases probabilities of leakages and pipe bursts. The high pressure above thresholds was reduced to an allowable range by using PRVs. However, the location, set-up, and number of PRVs were found as important parameters to be optimized for effectiveness in reduction of real loss. In this study, the real loss had higher percentage of contribution to NRW so its reduction may significantly decrease the total NRW. The NRW analysis showed high NRW of 50% in a DMA compared to 37% for the entire network. Particularly, extremely high percentage of about 87% of NRW contributed from leakage component. This indicates that despite general strategies to address water loss reduction, the modelling and analysis of zone by zone can be useful in prioritization of approaches to solve the problem. Due to dynamic of water demand caused by urban sprawl, regular nodal pressure assessment and flow modifications found to be a solution to control demand variations. This is cautionary for change of common practiced fixed system design approach to stage or flexible system designs. The applied approach for performance analysis and modelling of water networks is of high importance for proper management and improvement of the systems. This can open the way water utilities operate the water distribution networks that may result in significant benefits in reduction of NRW, improving performance of the system and controlling stochastic demand growth for present and future sustainability of the provision of the services. Also, the reduced water loss may save money used foe chemicals in its treatment and energy for water supply, thus lowering operation costs and store water for future consumptions.

5.2 Recommendations

In this study, the measurements and hydraulic analysis have revealed the shortcomings of the system. The pressure and velocity parameters have their recommended thresholds that have to be met for satisfaction of service delivery since they have implications in the performance of the WDN. Its monitoring and controlling is crucial in operations and maintenance of the distribution network. The system pressure management is a recommendable practice for all water utilities so as to decrease chances of high number of leakages and pipe bursts. The low range velocities within the system hinders free water circulation. This may cause stagnant water to deteriorate its quality. More studies are required to investigate the water quality status. As it has been observed the sizes of pipes were bigger compared to demand, so water is mostly not flowing, rather is stored in the pipes. Most water utilities use fixed design approach for a specified design horizon. However, for fast-growing cities with random demand growth, the appropriate approach suggested is flexible and stage designs.

The NRW has been a challenge facing many of the water utilities in Tanzania and, generally, most of the whole world. The developing countries have become worse due to some other factors including limited resources and behaviour of handling WDN management. Most of their fast-growing cities comprised of unplanned WDN, which complicates most the optimization and management. Despite the challenge but there is low initiatives to do researches in investigating the NRW. Less information regarding the hydraulic behaviour and conditions of the system is available. In order to address this challenge, the field measurements and hydraulic parameters analysis are recommended. The efficiency of prioritization of the small areas in DMAs to focus on strategies for NRW reduction might be one of the parts to work on in most of the water utilities, thus the zonal by zonal NRW analysis is emphasized. The main component contributing to the NRW was real loss than apparent loss. This enhances decision making on activities and strategies to reduction of the NRW programs.

The pressure is a dynamic parameter in the performance of the WDN. It can be affected by increasing the demand in the system or changing the network topology. It requires appropriate attention in controlling and monitoring. The most common technology used in Tanzania is Break Pressure Tank (BPT) than PRV. The BPT reduces the pressure to zero without regulating options. This is the limit when there are changes that need increase of pressure. The adjustment of pressure by PRV is more effective as it can be installed depending on the demand variations in the system. A thorough investigation is required to do costs benefit analysis between supplying water with high pressure and reduce pressure to the low elevation areas or supplying with less pressure and install booster points to convey water hilly areas. This is important as in the same pressure zone some parts have higher pressure than others. In order to balance pressure within the adequate ranges the sub-zones are suggested as solutions to ensure only required demand and pressure are provided in a respective area.

As unplanned distribution networks encounter random demand growth, they require regular review and modification of the demand and pressure in the system. The pressure and flow thresholds are the controls for reliable services. The maximum, minimum and service residual pressure heads in the system can give guidance to modify the parameters and simulate future conditions. Forecasting the junctions which do not require more withdrawal should be determined. Increased supply to vulnerable junctions is suggested as the best solution which is more sustainable. The algorithm for hydraulic analysis by considering junction residual pressure and flow modification may be important to be developed to ensure spatial and temporal system sustainability.

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RESEARCH OUTPUTS

(i) Publication

Shushu, U. P., Komakech, C. H., Arhin, D. D., Ferras, D., & Kansal, L. M. (2021). Managing non revenue water in Mwanza, Tanzania: A fast growing sub-Saharan African City. *Scientific African*, 12(2021), 1-10.

(ii) Poster Presentation