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**QUANTIFYING WATER LOSS COMPONENTS AND CONDUCTING PERFORMANCE
INDICATOR-BASED ASSESSMENT FOR ADDIS ABABA WATER DISTRIBUTION
SYSTEM**

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**A Thesis submitted to The School of Civil and Environmental Engineering Presented in
Partial Fulfillment of the Requirements for a Degree of Masters of Science in Water Supply
and Environmental Engineering (Civil Environmental Engineering Stream)**

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DECLARATION

I, Fikirte Girma Mekonnen, declare that this research is my work, and additional sources used have been properly and fully acknowledged using references. This thesis has not been submitted before for any degree or examination in any other University.

I am responsible for the research and its articulation alone. In no way do any of the persons mentioned in the acknowledgment bear any direct responsibility for this work.

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This is to certify that the thesis prepared by **Fikirte Girma Mekonnen**, entitled; **Quantifying water loss components and conducting performance indicator-based assessment for Addis Abeba water distribution system** and submitted in partial fulfillment of the requirement for the degree Master of Science (Civil and Environmental Engineering, Major of Water Supply and Environmental Engineering) complies with the regulations of the university and the meets accepted standards concerning originality and quality.

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ABSTRACT

Water loss in urban water distribution systems is a significant global challenge, leading to wastage, revenue losses, and contamination risks. The Addis Ababa Water Distribution System (AAWDS) faces similar challenges, a significant portion of the water supplied to people doesn't reach its intended users. Despite the critical importance of mitigating water loss, assessment practices and understanding of different causes and factors influencing the amount and type of water loss in a distribution system is limited. This study aims to assess and quantify total water losses in the AAWDS, and evaluating system performance. The research examines both real and apparent losses, with a focus on metering accuracy degradation rates and failures, especially in DN 15 mm customer meters, which constitute a large proportion of utility meters. The study identifies a high meter failure rate (0.59%/year) in the system, with water losses due to DN 15mm meter failure estimated at 0.53% of the annual non-revenue water (NRW), equivalent to $470,081 m^3$ /year. Using comparative billing analysis methodology, the research determines meter accuracy degradation rates for different meter models and calculates annual water losses due to meter inaccuracies, totaling US\$1.5 million per year. Additionally, the study adopted two techniques, Minimum Night Flow Analysis and Calibration Process Water GEMs V10 Calibrator Software, were used to estimate leakages in district-metered areas situated around Kasanchis area, revealing a daily water loss $756 m^3$. Leakage identification in the water distribution system of the DMA resulted in shortlisting of 5 points. The research also proposes 12 performance indicators for evaluating the system's efficiency, with some successfully tested on a branch utility of AAWSA. The findings emphasize the need of integrated water meter management (IWMM), leakage detection with calibrator software, and the utility's use of appropriate water loss performance indicators.

Key words: *Non-Revenue Water, Water loss, Apparent loss, Leakage, District Meter Area, Performance Indicators.*

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ABBREVIATIONS

AAWDS	Addis Abeba Water Distribution System
AAWSA	Addis Ababa Water and Sewerage Authority
AWWA	American Water Work Associations
AZP	Average Zonal Pressure
CARL	Current Annual Real Loss
CP	Critical Point
DMA	District Metering Area
EPS	Extended Period Simulation
FAVAD	Fixed and Variable Area Discharge
GIS	Geographical Information System
ILI	Infrastructure Leakage Index
IWA	International Water Association
IWMM	Integrated Water Meter Management
MNF	Minimum Night Flow
NHM	Network Hydraulic Modeling
NRW	Non-Revenue Water
PI	Performance Indicators
UARL	Unavoidable Annual Real Loss
UNICEF	United Nations Children's Fund
WDS	Water Distribution System
WLA	Water Loss Assessment
WLM	Water Loss Management
WHO	World Health Organization

1 INTRODUCTION

1.1 Background of the Study

Water is the most precious natural resource and lifeblood of our planet. It is the most crucial for our survival, peace, and prosperity. The world's freshwater resources are under increasing pressure, Growth in population, increased economic activity, and improved standards of living lead to increased competition for and conflicts over the limited freshwater resource (Mekonnen & Hoekstra, 2016). Water management focuses on generating new sources rather than managing existing ones. Moreover, water management is usually left to top-down institutions, the legitimacy and effectiveness of which have increasingly been questioned. Thus, the overall problem is caused both by inefficient governance and increased competition for the finite resource (Maliva & Missimer, 2012).

Another challenge is Ethical urban planning and infrastructure development have been seriously violated due to random urban developments. Furthermore, water distribution network expansions have been done without proper consultation resulting in serious systems pressure deficiencies, systems head losses, poor quality water distribution network materials and poor water supply installation workmanship, and unpredicted future demand (Eugine, 2015). The challenges of high water demand are exacerbated by the fact that there has been very limited capital investment in water infrastructure development especially in developing countries. Thus, the water demand pressures highlight the need for a paradigm shift to utilize water resources as efficiently as possible. Without effective actions, scarce water resources will be diminishing even faster soon due to increasing demand for water. For long-term sustainability, and the fulfillment of wide-ranging options for human needs, the United Nations (UN) proposed the implementation of Integrated Water Resource Management (IWRM). Effective integrated water resources management (IWRM) is thus crucial to ensure that scarce water is supplied efficiently, where and when it is needed (Biswas, 2004). The Global Water Partnership (GWP) defined IWRM as “a process that promotes the coordinated development and management of water, land, and related resources to maximize the resulting economic and social welfare equitably without compromising the sustainability of vital ecosystems.

Access to a safe and adequate water supply is a universally recognized human right, which is a fundamental human need. Adequate water supply may be defined as having reasonable access to

safe water supply, performance of water utilities can be assessed by many factors including accessibility and reliability of water supply, affordability of services, and customer satisfaction. However, according to WHO and UNICEF Joint Monitoring Programme of UN MDGs, 884 million people in the world do not have access to improved water supply sources, almost all of them in the developing countries of Africa, Asia, and Latin America. Ironically, significant amounts of safe drinking water continue to be lost in the urban water distribution systems of developing countries. According to the World Bank, nearly 45 million m³ of water is lost daily as leakage in water distribution systems of developing countries – enough to serve about 200 million people. Furthermore, the World Bank estimates that close to 30 million m³ of water is delivered every day to customers but not invoiced due to metering inaccuracies, theft, billing errors, and corruption by utility employees. This costs water utilities in developing countries about US \$6 billion every year.

Water losses not only represent economic loss and wastage of a precious scarce resource but also pose public health risks. Every leak is a potential intrusion point for contaminants in case of a drop in network pressures. Leakage also often leads to service interruption and customer complaints, is costly in terms of energy losses, and increases the carbon footprint of the service provider. These problems are likely to be compounded in the future as a result of the widening gap between aging water supply infrastructure and investment, rapid population growth, poor management practices, poor governance, and more extreme events as a consequence of climate change. These unprecedented pressures coupled with diminishing water resources and increasing costs of supplying water, have led regulatory bodies and water service providers to consider seriously urban water demand management and conservation measures. The high water losses in water distribution systems present an excellent opportunity for “un-tapped” water resources that have already been treated to drinking water standards and could be recovered cost-effectively. Recovering water losses requires understanding why, where, and how much water is lost, and developing appropriate intervention measures (H. E. Mutikanga, 2012) & (H. E. Mutikanga et al., 2009).

Globally, water distribution losses represent a significant issue with a wide range of impacts seen in different countries. These losses may be as low as 3% of the total system input volume in developed countries, indicating efficient water management systems. However, in developing

countries, water losses can be as high as 70% due to factors like ageing infrastructure, poor management practices, and data-deficient networks. These extreme losses can be attributed to both real losses (such as leaks) and apparent losses (like inaccurate meter readings or unauthorized consumption). Developing countries often lack the necessary resources to reduce these losses effectively. An estimation suggests that reducing water losses by half in developing countries could lead to more than 22 million cubic meters per day of saved water, enough to serve over 100 million people. In terms of revenue, water utilities could potentially recoup about US \$3 billion each year.

Water utilities in developing countries are struggling to provide customers with a reliable level of service, often via aged water distribution infrastructure, with data-poor networks and restricted budgets. Understanding the condition of the water distribution system is a key factor in minimizing water losses. Although real-time in-service pipeline inspection is the direct ideal method, it is costly and out of reach for most water utilities in the developing countries. Alternative indirect assessment of water distribution systems based on the water balance and performance indicators seem to be more practical (H. E. Mutikanga et al., 2013). The International Water Association (IWA) and the American Water Works Association (AWWA) have developed a standard water balance methodology and an array of performance indicators for water loss management. Whereas the IWA/AWWA water balance methodology and performance indicators provide a good foundation, they are insufficient and not directly applicable to water distribution systems in developing countries. They require large amounts of reliable data that is costly and hardly generated by the resource-constrained water utilities of developing countries. Therefore, each utility must examine its operational processes and practices, infrastructure, organizational culture, supporting data, technology, and available resources and develop performance measures most appropriate for its local working environment.

Different types of water losses exist, influenced by various causes and factors within a water distribution network. Without a comprehensive understanding of these losses, finding effective solution to reduce them is challenging. Therefore, this study aims to achieve the following objectives.

Firstly, it seeks to provide a thorough understanding of water losses. Secondly, it aims to identify the causes of water losses in the distribution system. Thirdly, the study intended to demonstrate to select an appropriate performance indicator(PI) from IWA/AWWA guidelines to measure the efficiency of AAWSA water loss management. It is important to note that not all PIs are universally relevant, as their usefulness depends on factors such as data availability, accuracy, and applicability to the specific water utility.

Finally, the study will recommend an effective water loss reduction measures. Taking action against water losses is typically a worthwhile investment for water utilizes, as it can lead to significant cost savings.

1.2 Statement of the problem

The real loss is one of the major causes of the loss of water in a distribution system. Real Losses are physical water losses from the pressurized system, up to the point of customer metering. Studies indicate that, the volume lost through all types of leaks, bursts, and overflows alone is sufficient to supply approximately 200 million people in developing countries (Eugene, 2015) & (H. E. Mutikanga et al., 2009).

(Desalegn, 2005), study that 41 percent of water was found to be lost in the city of Addis Abeba. Leakage is observed as the main cause of water loss from the water distribution system, which is one of the components of real loss. As the study implies the cause of water loss is caused by poor operational practices, poor infrastructure, network characteristics, and bad management practices, in addition to the lack of different time scale data, periodical maintenance, and lack of motivation at the management level (Kumar Bhagat et al., n.d.).

To effectively manage real loss, it must be accurately assessed. Improving the efficiency of delivering water to customers requires a thorough understanding of the condition of the water distribution system (WDS) and addressing the problem of how much water is being lost, where, and why. In such cases, performance measures such as the water balance and performance indicators (PIs) should be considered. The infrastructure leakage index is a useful performance indicator for reducing real losses (Lambert et al., 1999). However, the challenge lies in the fact that most WDS performance measures widely used in developed countries, such as the unavoidable annual real losses (UARL) and the infrastructure index (ILI) (Lambert et al., 1999),

are questionable in the context of most developing countries. For the AAWDS, this is a research area that has not yet been addressed but is critical for a meaningful evaluation. Another research area that has yet to be thoroughly addressed is the Apparent losses in AAWDS. While extensive research has been conducted to address water losses in WDSs, little work has been done so far on apparent losses, which are assessed using rules of thumb. So far, no study has been done to examine the apparent losses in AAWDS. Water meter performance in WDSs of the AAWSA that are not so well managed and provide water irregularly is still not very well understood. In general, apparent loss management in Addis Abeba water supply systems are still in its infancy, and much work remains.

Clearly, knowledge gaps still exist as mentioned above in water distribution systems of Addis Abeba and this research seeks to bridge the knowledge gaps by determine and analysis the appropriate performance indicators (PIs) which is relevant and applicable for AAWDS and assessing and quantify leakage on district Meter Area (DMA) and apparent loss this research is going to focus.

1.3 Research Objective

1.3.1 General objective

The General Objective of this research is to assess and quantify water losses in Addis Abeba water distribution systems, as well as to identify appropriate performance indicators to evaluate the efficiency of Addis Abeba water distribution system, with a special focus on water loss reduction to improve water distribution system efficiency and validate its effectiveness through application to AAWSA branches.

1.3.2 Specific Objective

- Ø Asses and quantify the present status of water loss
- Ø To identify the main causes of water loss within the distribution system
- Ø To determine and Analyse relevant performance indicators
- Ø To recommend appropriate water loss reduction measures

1.3.3 Research Questions

Table 1.1: Research Question

Research Question
Ø How much water is being lost due to Apparent Losses from Water distribution system?
Ø How much Water is being lost due to Leakage from the Water distribution system?
Ø What are the causes of water losses in the water distribution system?
Ø What are an appropriate water loss performance indicators (PIs) in order to evaluate the performance of water supply utility (AAWSA)?
Ø What are the possible solutions to reduce the water loss?

1.4 Scope

The scope of the study is assessing and quantifying total water losses in Addis Abeba water distribution system (AAWDS) by disaggregating real and apparent losses in order to understand the nature of the water loss and evaluate the efficiency and effectiveness of performance water loss management for Addis Abeba water distribution system by applying relevant IWA/AWWA performance indicators approach.

1.5 Significance of the study

This study has significance for Addis Ababa Water and Sewerage Authority. Performance assessment of water utility based on performance indicators help on developing technical strategies for water loss reduction, in order to decrease the water availability challenge which is worsened by high volume of water loss and to enhance service delivery to meet international standards.

1.6 Thesis outline

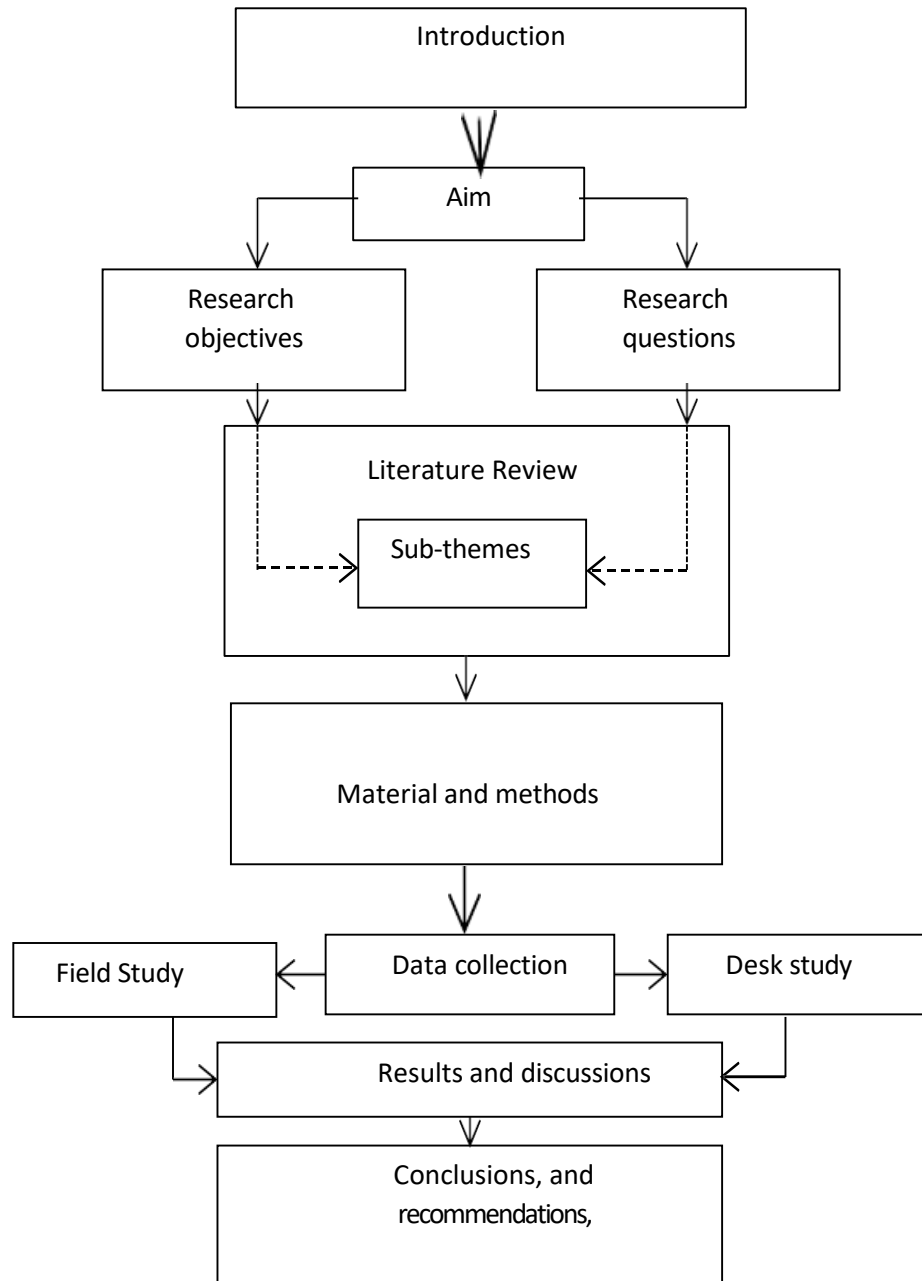


Figure 1 Schematic representation of the thesis outline

2 LITERATURE REVIEW

2.1 An overview of global water distribution losses

Water loss in the distribution system has been reported, globally. Water loss management strategies have been developed in many countries over a long period, since the development of water reticulation systems. However, in many developing countries the concept of water loss management has received very little attention compared with the severity of the problem of water loss. Water management is a top responsibility for all water utilities due to its global importance.

urban water distribution systems can be referred to as "buried and forgotten" because they are infrastructure that is typically underground and out of sight. Because of this, these systems can often be neglected or overlooked until significant problems like water leakage or system failure occur (Lambert and Hirner 2000). This can be more pronounced in developing countries where funding for infrastructure maintenance and upgrade is limited. Such neglect can lead to significant water losses, inefficiencies, and poor service delivery. Therefore, maintaining regular investments and attention to these systems is crucial for sustainable water management (H. E. Mutikanga, 2012).

Generally, non-revenue water (NRW) is the difference between SIV and water delivered to consumers and billed (revenue water). NRW is made up of water losses (real and apparent losses) and authorized unbilled consumption such as water for firefighting and flushing mains. The amount of water loss is an indicator of a WDS's operational efficiency. High levels of Water loss are a sign of poor WDS governance and physical condition, and the costs of system inefficiency are passed on to users via excessive water tariffs.

A utility cannot operate efficiently if it does not realize all its revenue due to water losses. According to a World Bank report, NRW from WDSs is predicted to be 48 billion m³ per year, costing water utilities approximately \$14 billion per year (Kingdom et al. 2006). According to the same report, developing countries account for around 55% of global NRW by volume. The provision of appropriate water supplies to a fast-growing population in the face of such substantial water losses will continue to be a serious concern for many countries across the world.

According to United Nations World Water Development Report (2023), and WHO/UNICEF (2023) Around 2 billion people around the world do not have access to clean and safe drinking

water, with the vast majority living in developing countries. This issue is anticipated to be worsened by the region's fast-growing urban population. 56% of the world's population, or 4.4 billion inhabitants live in cities. This trend is expected to continue, with the urban population more than doubling its current size by 2050, at which point nearly 7 of 10 people will live in cities. (World Bank,2023). The harsh reality is that the world's water supplies are finite and limited, and can no longer maintain this rate of increase unless handled carefully. The high water losses in WDSs present “un-tapped” water resources that can be recovered cost effectively. These untapped and wasted resources are already treated to drinking water standards and energized to provide adequate pressure to reach consumers.

Water losses do not only have economic and environmental dimensions but also have public health and social dimensions. In particular, leakage often leads to service interruption and may cause water quality deterioration through pathogen intrusion (Karim et al., 2003; Almandoz et al., 2005). The contaminated water can infiltrate the piped network through small leaks and cracks when the outside water pressure becomes greater than the water pressure within the pipe.

2.1.1 Water losses in developed countries

In many aspects, water loss management scenarios in developed countries differ from those in developing countries. The key difference is in the reaction plans and response sensitivity of these countries' water utilities and administrations. Water losses in developed countries have been reduced to generally acceptable and manageable levels (Farley & Trow, 2015).

According to the sources, developed countries face significant water distribution losses. In Greece's Larissa city, non-revenue water has been estimated at 34% of the system input volume. Similarly, in Turkey, approximately 60% of total water losses are physical losses, while the remaining 40% are accounted for apparent losses (Surendra et al., 2021). Water distribution systems in the Netherlands are considered among the most efficient globally, with water losses reported in the range of just 3-7% of total system input (Beuken et al. 2006). In the USA, an estimated 22 million cubic meters of water is lost per day or categorized as public use/loss, with an average non-revenue water rate of 15%, but a range from 7.5% to 20% (Grigg 2007). Water loss due to main breaks is noted as significant and is comparable to annual flood losses, which cause over \$2 billion of property damage.

Water distribution losses continue to be a significant issue in developed countries, impacting the overall efficiency of water utilities. In the UK, despite being perceived as a leader in leakage management, approximately 20-23% of delivered water is lost through leakage, with little variation over the past decade (OFWAT 2010). This highlights the challenges faced by water utilities in effectively managing water distribution losses. Italy also faces substantial NRW levels, ranging from 15 to 60% with an average of 42% (Fantozzi 2008). Furthermore, Portugal experiences NRW averages of 34.9%, varying from less than 20% to more than 50% (Marques and Monteiro 2003). In Australia, for a data set of 10 water systems, NRW varied from 9.5% to 22% with an average of 13.8% (Carpenter et al. 2003). In Canada's Ontario province, leakage varies from 7% to 34% (Zechner 2007). Despite these figures, apparent losses (due to billing inaccuracies or unauthorized use) are also important. While these may appear low in terms of percentages, the financial impact can be significant, making it a critical area for water utilities in developed countries to address. For instance, apparent losses in England and Wales are estimated at 118 million m³/year.

In developed countries, the management of water distribution losses is crucial for ensuring efficient and sustainable water supply systems. Research has shown that the cost of lost water due to leakages, metering errors, and non-billed consumption in developed countries is approximately US\$39 billion annually. Water utilities in developed countries have made efforts to reduce water losses and improve the efficiency of their systems. These efforts have included implementing advanced technologies, such as smart meters and remote monitoring systems, to detect and identify leaks more effectively.

2.1.2 Water losses in developing countries

Many developing-country water distribution systems operate under intermittent conditions (WWAP, 2014). As a result, the efficiency of these countries' water supply is jeopardized. Aside from the issues connected with intermittent water supply situations, water losses in developing countries have reached an alarming rate, with non-revenue water levels exceeding 60% in many of these countries. Political interferences and institutional reluctance to change characterize the slow progress in water loss reduction in developing nations (Savanije and van der Zaag, 2002; Gumbo and van der Zaag, 2002). One of the key reasons for the poor pace is that utilities and water

supply companies have not been reinvesting profits in network restoration. The majority of revenue collected by water operators is diverted to other uses rather than being used to maintain and upgrade water distribution systems.

The Asian Development Bank finds NRW in Asian cities ranging from 4.4% of total water supply (PUB, Singapore) to 63.8% (Maynilad, Manila) (ADB 2010), with apparent losses accounting for 50-65% of NRW (McIntosh 2003). NRW of 40-55% of supplied water has been recorded in Latin American water companies (Corton and Berg 2007). Water losses in Brazil average 39.1% of water supplied, equivalent to over 5 billion m³ of water lost each year (Cheung and Girol, 2009). NRW statistics in Africa have ranged from 5% (Saldanha Bay, South Africa) to 70% (LWSC, Liberia) (WSP 2009). Countries in the Middle East and North Africa Region (MENA) such as Tunisia and Algeria face absolute water scarcity, with less than 500 m³/person/year of freshwater (Baroudy 2005). Kenya falls below the freshwater water poverty level in East Africa, which is described by experts as 1,000 m³/person/year (Qdais 2003). Tanzania and Uganda will be approaching critical levels by 2025 (WRM 2005).

According to a World Bank study, nearly 45 million m³ of water is lost daily due to leakage (enough to serve nearly 200 million people), and nearly 30 million m³ of water is delivered to customers but not paid for due to metering inaccuracies, theft, and corrupt utility employees, costing water utilities approximately US \$6 billion per year (Kingdom et al., 2006). This is inappropriate in a situation where water providers are desperate for additional revenue to expand services to the poor and water is tightly rationed, and heavily wasted. This is anticipated to be exacerbated by the high rate of infrastructure deterioration, resulting in a higher loss of treated drinking water. Extreme shortage situations are likely a result of poorly managed urban WDSs combined with increasing global change pressures (urbanization, climate change, population expansion). To escape the impending water scarcity, developing countries must undergo a profound paradigm shift in the way water resources are managed.

2.2 Water Distribution Challenges Faced by Developing Countries

According to WHO and UNICEF (2010), 343 million people in Africa, 477 million in Asia, and 38 million in Latin America and the Caribbean lack access to safe drinking water. The urbanization tendency is especially pronounced in Africa. According to Cohen (2006), Africa's urban population is expected to more than quadruple, from 295 million in 2000 to 1.5 billion in 2030,

with slums housing more than 72% of the urban population. Meeting the United Nations Millennium Goal 7 (goal 10) of halving the number of people without adequate water and sanitation by 2015, amidst substantial water losses in developing-country WDSs, will remain a major problem for the twenty-first century. In the broader context of urban water demand management, public water suppliers bear a significant responsibility to manage water responsibly and efficiently on the supply side by reducing water losses from distribution systems, which complements customer-side demand management of water efficiency. This will help to accelerate service delivery while also ensuring the sustainability of urban water systems in developing country cities.

Other major challenges of WLM include aging infrastructure, inadequate resources, poor governance issues, inadequate asset management, poorly designed WDSs, insufficient reliable data for WDS performance evaluation, and intermittent supply. The prospects lie in increasing capacity building of water utility employees, research, performance-based contracting, emerging new equipment and technology for leak detection, and increasing dissemination of emerging “state-of-the-art” tools and methodologies for water loss reduction and performance improvement of utility water services (H. E. Mutikanga et al., 2009); (H. E. Mutikanga, Sharma, et al., 2011a); (Alegre et al. 2006); (Arregui et al. 2006); (AWWA 2009); (Fanner et al. 2007b); (Farley and Trow 2003); (Thornton et al. 2008); (Wu et al. 2011)).

2.3 Water loss in Ethiopia

Ethiopia is the most populated and largest country in the Horn of Africa, with a total land area of 1,000,000 Km² (386,102 sq. miles) and a population density of 127 per Km² (328 people per mi²). Ethiopia's population is equivalent to 1.57% of the global population. Ethiopia's current population is predicted to be 127,891,986 according to Worldometer 2023.

Ethiopia is a federal state divided into twelve regions and two "chartered cities" (Addis Abeba and Dire Dawa). The twelve regions, each with a population ranging from 200,000 to 40 million people; 68 zones, each with a population ranging from 100,000 to a few million people. 770 Districts, each with a population of 10,000 to more than 300,000 people, the biggest of which are found in urban regions; several thousand Kebeles, the smallest administrative entities in Ethiopia, each with a population of a few thousand people.

Ethiopia water and sanitation sector, the division begins at the federal level with the Ministry of Water Resources (MWR), then moves to the regional level with Regional Water Bureaus (RWBs), and finally to the local level with town water boards and Woreda (Spoon, 2010). Since 1995, the government has implemented a water and sanitation policy that calls for further decentralization of municipalities.

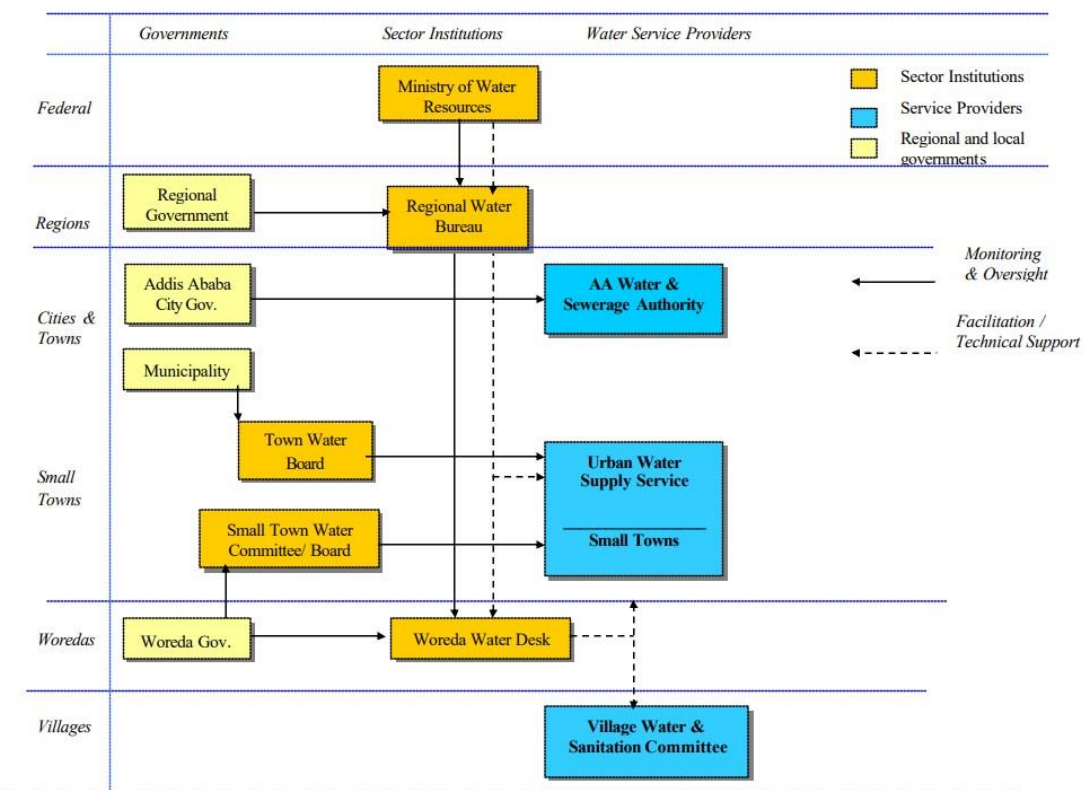


Figure 2.1 Ethiopia water and sanitation sector institutional arrangements, *Source: (WSP, 2004)*

As previously stated, there are four water service providers serving the entire country. According to (WSP, 2004), there is only one Metropolis with a population of more than 2 million that is managed by Addis Ababa Water Supply and Sewerage Authority (AAWSA). The Urban Water Service Unit (UWSS) manages the 10 larger towns with populations ranging from 80,000 to 250,000, while the 141 medium towns (10,000 to 80,000 inhabitants) are governed by the Urban Water Service Unit (UWSS). Small town services utility covers 491 small towns (2000 to 10,000 people) and 282 rural towns with populations less than 2000, with technical help from RWB/Zone.

According to the World Bank's collection of development indicators, Ethiopia's urban population was estimated to be 27, 959,125 in 2022 (22.66% of the total population). Addis Abeba is Ethiopia's largest city, with a population of 3,945,000 people (25 percent of the country's urban population). According to WHO, urban water coverage is predicted to be 84% (71.7 million), national coverage is 50% (19.2 million), and rural coverage is 41% (52.5 million). Water loss in Ethiopian cities is much higher than the World Bank's permissible limit of 25% (Beker & Kansal, 2023).

Water distribution losses in developing countries like Ethiopia are a significant challenge, with high levels of non-revenue water contributing to these losses (Zewdu, 2014; Siraj, 2016; Ayalew, 2021). Factors such as ad-hoc maintenance, insufficient financial resources, and poor operations and maintenance in the pipelines are identified as key contributors to these losses (Zewdu, 2014; Ayalew, 2021). The impact of these losses is particularly severe in Ethiopia, where access to safe drinking water and improved sanitation is among the lowest in the world (Siraj, 2016).

A range of studies have highlighted significant water distribution losses, with Axum town experiencing a high loss of 39.1% due to ad-hoc maintenance and insufficient financial resources (Zewdu, 2014). Similarly, Adwa town has a 43.87% water loss, with the main causes being leakage and illegal connections (Brhane, 2019). In Debre Berhan town, NRW of 37.4% of Supplied water was observed, among this 6.78% real loss level and 30.62% of apparent loss have been reported (AAiT 2017 G.C., 2017). In Debre Markos town, 40 % of NRW was reported, and about 33% of the total system loss are real losses and 7% apparent losses (Melaku & Geremew, 2015). In Hosanna (Wamolo, 2020) and Adama (Abduro, 2015), respectively, 29.48% and 34% of the NRW of water delivered were recorded. In Gimbi town, non-revenue water accounts for 55% of production, and 60% of non-revenue water is estimated as a real loss (Lencho, 2020). In Finote Selam, the water loss of the town was 32.18% of the total water production. The apparent loss covers 9.18% while, the real losses cover 23% of the total losses (Maulidina, 2019).

These losses are exacerbated by the use of unprotected and unsafe water sources, particularly in rural areas (Siraj, 2016). The situation is further complicated by service failures in rural water supply systems, which are influenced by the lack of uniformity in implementation approaches and institutional and organizational incapability (Behailu, 2017). The need for effective water loss

management strategies in the country is emphasized, with a focus on reducing leakage and improving the financial viability of water utilities, and enhancing institutional capacity to address the issue of water distribution losses in Ethiopia.

2.3.1 Water losses in Addis Abeba

Addis Abeba, the capital city of Ethiopia is one of the developing cities suffering a high shortage of water as well as high water loss. Nearly 50% of the water produced is lost at different levels of the distribution system before reaching the consumer (Desalegn, 2005).

Addis Ababa Water and Sewerage Authority (AAWSA) is responsible for the water supply and sewerage services in the city. The city water supply covers 56% of the city area (300 km²). In order to deal with water losses, Farley (2003) indicated that a water loss management strategy must be put in place. Farley further explained that the key to developing a strategy for management of water losses is to gain a better understanding of the reasons for the losses and the factors which influence its components. Techniques and procedures could then be developed, and tailored to the specific characteristics of the network and local influencing

Factors, to tackle each of the components in order of priority. Fantozzi et al. (2006) came up with the following steps on how to begin implementing a water loss reduction strategy:

Step 1: Assess your losses in volume terms, using the best practice international standard IWA annual Water Balance

Step 2: Identify ‘how are we doing?’ using best practice performance indicators (PIs) as outlined in the IWA blue pages by Lambert and Hirner (2000)

Step 3: Analyze your data and identify your priorities and strategy

Step 4: Get started and learn as you progress

Based on the outlined steps, most utilities that have not started water loss management practices can get started in order to realize the benefits of reduced leakage.

2.4 Water Loss Management

Water losses occur in every water distribution network (WDN) in the world. For economic and technical reasons, it has to be accepted that real water losses cannot be entirely eliminated. Nevertheless, there has been a large increase in the knowledge and development of state-of-the-

art equipment, allowing us to manage water losses within economic limits (American Water Works Association, 2016); (Alegre H., Hirnir W., Baptista J., 2006); and (Lambert et al., 2000).

Definition and terminology

The definitions and terminologies used in this study for water loss assessment are those that have been developed by the International Water Association (IWA) task forces on PIs and water losses (Alegre et al., 2016); and adopted by the AWWA (AWWA 2020). The terminologies used are shown in table below:

Table 2.1 The IWA/AWWA water balance methodology

System Input Volume	Authorised Consumption	Billed Authorised Consumption	Billed Metered Consumption (including water exported)	Revenue Water
			Billed Unmetered Consumption	
		Unbilled Authorised Consumption	Unbilled Metered Consumption	Non- Revenue Water (NRW)
			Unbilled Unmetered Consumption	
	Water Losses	Apparent* Losses	Unauthorised Consumption	
			Metering Inaccuracies	
		Real* Losses	Leakage on Transmission and/or Distribution Mains	
			Leakage and Overflows at Utility's Storage Tanks	
	Leakage on Service Connections up to the measurement point			

The elements of the water balance are defined as:

System input volume: is the volume of water input to a transmission system or a distribution system

Authorised consumption: is the volume of metered and/or unmetered water taken by registered customers, the water utility's own uses, and uses of others who are implicitly or explicitly authorized to do so by the water utility; for residential, commercial, industrial and public-minded purposes.

Revenue water: Those components of System Input Volume that are billed and have the potential to produce revenue.

Non-revenue water: Apparent Losses + Real Losses + Unbilled Metered Consumption + Unbilled Unmetered Consumption. This is water that does not provide revenue potential to the utility.

Water losses: It can be expressed as the difference between non-revenue water (NRW) and unbilled authorized consumption, and consists of apparent and real losses.

2.5 Real Losses in Water Distribution System

Real losses are physical water losses from the pressurized system, up to the point of customer metering. The volume lost through all types of leaks, bursts, and overflows depends on frequencies, flow rates, and average durations of individual leaks (Lambert et al., 2000).

Real losses can be classified according to (a) their location within the system and (b) their size and runtime (Fallis et al., 2011)

Location

Leakage from the transmission and distribution mains: may occur at pipes (bursts due to extraneous causes or corrosion), joints (disconnection, damaged gaskets), and valves (operational or maintenance failure) and usually have medium to high flow rates and short to medium runtimes.

Leakage from service connections up to the point of the customer meter: service connections are sometimes referred to as the weak points of water supply networks because their joints and fittings exhibit high failure rates. Leaks on service connections are difficult to detect due to their comparatively low flow rates and thus often have long runtimes.

Leakage and overflows from storage tanks: are caused by deficient or damaged level controls. In addition, seepage may occur from masonry or concrete walls that are not watertight. Water losses from tanks are often underestimated and, though easy to detect, repair is usually elaborate and expensive.

(a) Size and runtime

Reported or visible leaks: primarily come from sudden bursts or ruptures of joints in big mains or distribution pipes. Leaking water will appear at the surface quickly depending on water pressure, leak size as well as on soil and surface characteristics. Special equipment is not required to locate the leak.

Unreported or hidden leaks: by definition have flow rates greater than of the order of 250 l/h at 50 m pressure but due to unfavorable conditions do not appear at the surface (Farley, 2001). The presence of hidden leaks can be identified by analyzing trends in water consumption behavior within a defined water supply zone. A wide range of acoustic and non-acoustic instruments is available to detect unreported leaks.

Background leakage: comprises water losses with flow rates less than of the order of 250 l/h at 50 m pressure, which do not appear at the surface. These very small leaks (seeping or dripping water from leaky joints, valves or fittings) cannot be detected using acoustic leak detection methods. Therefore, it is assumed that many background leaks are never detected and repaired but leak until the defective part is eventually replaced. Background leaks often cause a major share of real water losses due to their great number and their long runtimes.

2.6 Leakage Management

Real losses typically constitute a major proportion of total water losses. Leaks of different types (holes, longitudinal and circumferential cracks, leaking joints, etc.) in the distribution network's pipes are the main factor in real losses (Lambert et al., 2000).

Leakages in water delivery systems are a constant occurrence that will worsen if not addressed (Mathis et al., 2008). However, a leakage may continue to grow until it reaches a point where it stabilizes (McKenzie et al., 2002). In this regard, all water utilities must provide system maintenance and upkeep functions, including adequate leakage management, to avoid an increase in leakage rates.

2.6.1 Integrated Leakage Management Techniques

Leakage management involves assessment, detection and control (H. E. Mutikanga et al., 2009). An integrated leakage management technique comprises active leakage control, asset management, pressure management and Speed and quality repair (Pike, 2007). Such factors affect the management of leakages. Unless all the four leakage management activities (indicated by the arrows) in figure 2.2 are carried out to an appropriate extent, the current annual real losses (CARL) would increase as significant parts of the water distribution system (WDS) deteriorate further. In order to effectively managed it must be accurately assessed.

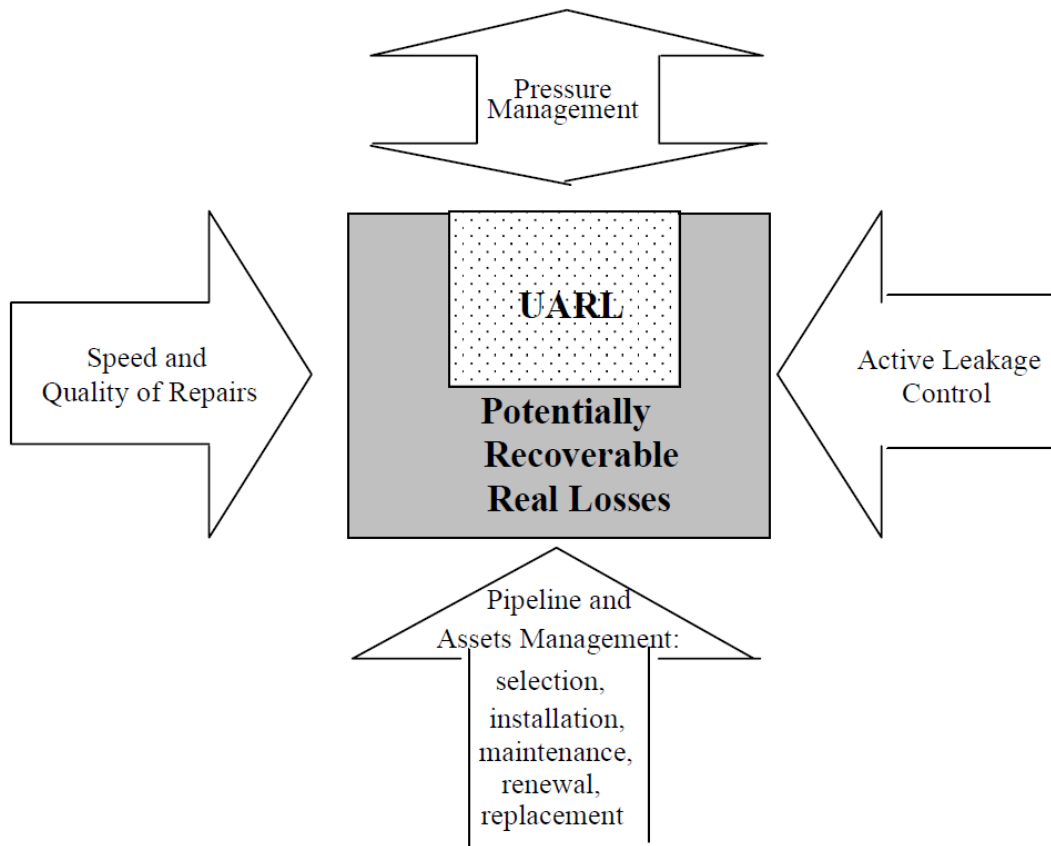


Figure 3 Integrated leakage management technique

2.6.2 Leakage Assessment Methods

Leakage assessment, often known as a water audit, aims to estimate the amount of water lost in the system analysed without concern where the leaks are actually located. The assessment methods that have been widely used for leakage assessment categorized as follows:

2.6.2.1 Mass (or volume) balance methods (Water balance/audit)

A water balance aims to track and account for every component of water that is added to and subtracted from a water supply system within a defined period of time. (Charalambous et al., 2002). The amount of water lost in a distribution system can be quantified by conducting a water balance. This can be performed either on a system wide basis or at the DMA level. There are two main water balance methodologies used for quantifying the volume of water losses: (1) the IWA/AWWA “top-down” water balance methodology (Alegre H., Hirnir W., Baptista J., 2006) &

(AWWA 2009) and (2) the UK “bottom-up” MNF analysis (Farley & Trow, 2015) and (Lambert, 1994).

These water balance methodologies evolved from earlier works in the United States by Male et al. (1985) and the Water Research Foundation (Wallace 1987). The water balance is an effective tool for systematic accounting of water supply and consumption. The United Kingdom water balance differs from the IWA/AWWA methodology in terminologies used; for example, the term “apparent losses” is not used in the UK methodology, illegal water use is categorized under unbilled authorized consumption the focus mainly on leakage assessment, and in addition the UK methodology considers meter under registration as part of revenue water, thereby under estimating NRW (H. E. Mutikanga, Sharma, et al., 2011a).

Table 3 The UK water balance methodology

Distribution Input (DI)			
Water Taken (WT)			Distribution Losses (DL)
Water Taken (WT)		Distribution Operational Use (DOU)	Distribution Losses (DL)
Water Delivered through Supply Pipes (WDS)		Miscellaneous Water Taken (WTM)	Distribution Losses (DL)
Measured (WDSM)	Unmeasured Use (WDSU)	Unmeasured Supply Pipe Losses (WDSL)	Miscellaneous Water Taken (WTM) [Legally and illegal, Meter under-registration]
			Distribution Losses (DL)

(Source: Lambert, 1994)

2.6.2.1.1 Top – Down Water Balance Approach

The IWA developed the top-down annual water balance as a best practice method. The objective of top-down leakage assessment approaches is to estimate the leakage in a particular system by evaluating different components of the overall water balance, primarily the water consumed for different purposes. Despite the simplicity of a top-down type leak assessment, the leakage estimate obtained via this method is referred to as a crude estimate. Gathering such information helps to decide what the next step in leakage studies should be for a particular network but it does not help to bound potential leak areas. (Puust et al., 2010)

2.6.2.1.2 Minimum night flow (MNF) method/ Bottom-up approaches

The bottom-up assessment is a useful tool for crosschecking the real loss volumes obtained from the top-down water balance and the component analysis. Bottom-up real loss assessment can be carried out in two different ways: (a) 24 Hour Zone Measurement (HZM) or (b) Minimum Night Flow (MNF) analysis. HZM needs a temporary isolated area of the distribution network that is supplied from one or two inflow points only. In these areas, 24-h inflow measurements shall always be logged along with pressure measurement (Puust et al., 2010).

MNF analysis uses field test data to quantify the volume of real losses within the distribution network. A DMA is required in order to conduct MNF measurements. A DMA is a hydraulically discrete part of the distribution network that is isolated from the rest of the distribution system consisting of about 500-3,000 connections (Farley & Trow, 2015). They are mainly used for monitoring and leak detection in WDSs.

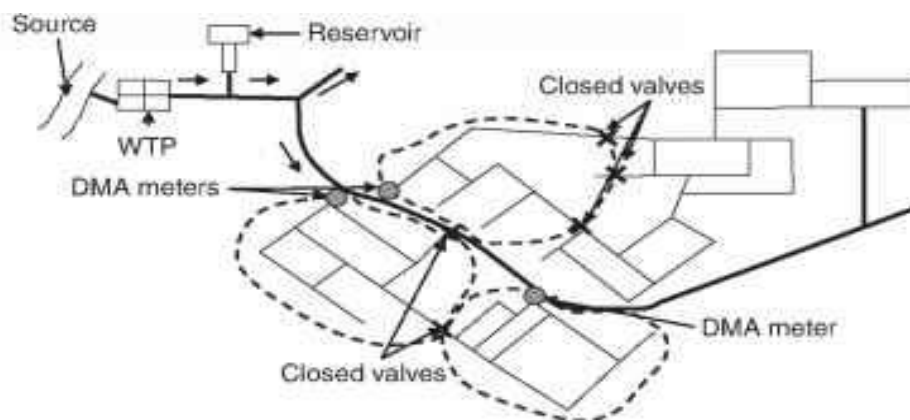


Figure 4 General DMA set up (Source: Adapted from IWA Water Loss Task Force)

Figure 4 General DMA set up The MNF in urban situations, usually occurring between 2 a.m. and 4 a.m., is the most meaningful piece of data as far as leakage levels are concerned. During this period, authorized consumption is at a minimum and, therefore, leakage is at its maximum percentage of the total flow. The estimation of the leakage component at minimum night flow is carried out by subtracting an assessed amount of legitimate night-time consumption for each of the customers connected to the mains in the zone being studied. Typically, in European and North American urban situations, about 6% of the population will be active during the minimum night-time flow period. This activity is typically to use a toilet and the water use is almost totally related to the flushing of WC cisterns (AL-Washali et al., 2018); (H. E. Mutikanga, Vairavamoorthy, et al., 2011). During night time water use is at its lowest and pressures in the network are relatively high and a significant amount of flow measured during the hour of MNF is likely to be leakage. To calculate the leakage at MNF time [$Q_L(t_{MNF})$], customer legitimate night time use must be accurately assessed and deducted from measured flow into the DMA [$Q_{DMA}(t_{MNF})$] at time of MNF (Equation 2.1). The hourly leakage rate (Q_L,t) throughout the day is then calculated by multiplying the Night-Day-Factor (NDF) with the leakage rate at MNF based on fixed and variable area discharge (FAVAD) principles that explain the pressure-leakage relationships (Lambert A.O., 2002).

$$Q_L(t_{MNF}) = Q_{DMA}(t_{MNF}) - \text{Legitimate Night-Time Uses} \dots\dots\dots(2.1)$$

$$Q_L(t) = Q_L(t_{MNF}) \times [P(t)/P(t_{MNF})]^{N1} \dots\dots\dots(2.2)$$

Where, $Q_L(t)$ is the leakage rate at the hour t ($t \neq t_{MNF}$), t_{MNF} is the MNF hour, $Q_L(t_{MNF})$ is the leakage rate at the MNF hour, $P(t)$ is the average hourly nodal pressure at the hour t ($t \neq t_{MNF}$), $P(t_{MNF})$ is the average hourly nodal pressure at the MNF hour, $N1$ is the pressure exponent. Studies have indicated that the $N1$ values range from 0.5 to 2.3 depending on type of leak and pipe material (Greyvenstein & Van Zyl, 2007).

MNF analysis was deemed to be doubtful and ineffective in Amman (Jordan), which was experiencing severe water rationing (Decker 2006). Decker (2006) proposed that the ideal strategy to using MNF approaches for leakage assessment would be to gradually establish limited DMAs that receive water on a regular basis. For more effective leakage evaluation, hybrid methods that

incorporate network hydraulic modeling, MNF, and top-down water balance approaches have been reported (Cheung et al., 2010).

Annual real losses can also be calculated from basic principles using component analysis (Lambert 1994; Wu et al. 2011). This method employs basic infrastructure data (mains length, number of service connections, etc.); an infrastructure condition factor (ICF) for background leakage; and average flow rates and runtimes of various types of leaks (background, reported, and unreported) on various WDS components (mains, service lines, reservoirs, etc.).

The water balance input data from flow instruments contains measurement errors and uncertainty. These uncertainties, as well as their propagation into output results, must be quantified and reported in order to provide confidence to water balance calculations. The purpose is to increase the accuracy of measurements and reported NRW values. Guidelines for quantifying uncertainty have recently been created (ISO 2008), and numerous scholars have attempted to quantify water balance uncertainties (Herrero et al. 2003); (H. Mutikanga et al., 2010) ; (Stent and Harwood 2000).

2.6.2.2 Network Hydraulic Modelling (NHM) simulations

WDSs are often very large and complex consisting of several kilo-meters of pipes of varying sizes and materials, storage reservoirs, pumps and various appurtenances. These systems are very difficult to understand and require large amounts of data for their analysis. NHM is one tool that has evolved over time to help engineers understand and manage WDSs. NHM involves using computer and mathematical models to predict the behavior of the WDS and are routinely used for operational investigations, planning tasks and network design purposes (AWWA 2005).

The numerous modeling applications available in the market are both freeware and commercial. Design software selection is based on factors such as data availability, budget, resources, applicability, compatibility, and project scope. Here are the basic tools for modeling water distribution (Sonaje & Joshi, 2015).

DisNet (2014) - This Software simplifies the process of creating water distribution networks while providing powerful and efficient capabilities. DisNet's key strengths are a user-friendly interface, high output accuracy, and detailed inputs. DisNet not only designs water distribution but also models stream hydrology, generates unit hydrographs, and establishes interrelationships. DisNet

is a CAD-integrated management application for simulating water distribution networks under various topography situations.

EPANET (2014) is a public domain software that can efficiently create any network. It offers several benefits, including water quality analysis, extended period modeling, and residual chlorine calculations for disinfection. It can also be utilized to upgrade or restore existing water supply systems. The program is public domain and referred to as EPANET 2.0, 2d-2w.

HydrauliCAD (2014) is an AutoCAD-based water distribution software that integrates with the EPANET hydraulic analysis application. Understanding the fundamentals helps develop a hydraulic model. HydrauliCAD allows for query-based addition and editing of hydraulic parameters such as head-loss, pressure, and flow in distribution networks. HydrauliCAD has an inbuilt pipe catalog with complete information on pipe materials, grades, and sizes. HydrauliCAD is used for both water distribution and fire-flow analysis. HydrauliCAD is a versatile software utilized by modelers, field specialists, and engineers.

H2Onet and H2Omap (2015) are commercial software that interface with GIS to build, analyze, and optimize many types of water distribution networks. This tool is useful for detecting and assessing leaks, analyzing fire flows and hydrants, and optimizing costs. The software's standout feature is its automated online SCADA interface. The connection with GIS software offers various vector and raster tools for geographical analysis, sampling, planning, evaluation, and assessment of both current and new water supply systems.

HYDROFLO3 (2015) is an enhanced version of the HYDROFLO series that may be used to design various distribution systems, including pumped, gravity, piped, and open channel flows. HYDROFLO3 simplifies conversions between Metric and SI units, validates existing networks, and allows for quick addition and adjustment of hydraulic parameters throughout the distribution network. HYDROFLO3 offers a unique feature of calculating pump hydraulic properties for forced flow systems. In addition to water distribution networks, HYDROFLO3 simulates treatment facilities, chemical dosing systems, industrial applications, and fire flow analysis. This software program helps create distribution networks and handle data effectively.

Pipe2014 the latest edition of KYPipe hydraulic modeling software, features a robust computational algorithm for fluids, primarily water. Pipe2014 can be used to design and select

pumps, valves, tanks, and pipes. It offers features such as pipe size and pump optimization. Pipe2014's user-friendly interface allows you significant flexibility in creating and optimizing distribution networks. Pipe2014 integrates with GIS and supports several image formats for planning distribution networks.

Synergi Water (2014) is a hydraulic modeling and simulation software suite with robust database management. It may be used to optimize existing distribution networks and build new ones. Synergi Water outperforms other public domain software by offering a versatile environment for detailed modeling, fast and accurate analysis of large systems with over one lakh components, water quality modeling, and complex system design with proper pump, valve, and tank arrangements. The system's flexibility with GIS and SCADA allows for seamless remote operations.

Water CAD V8i (2014) is a hydraulic modeling software that offers advanced graphical and profiling features, flexible data archiving and representation, and customizable GUI. Enhanced features include hydraulic and water quality analysis, steady state and extended period simulations, powerful data management, and AutoCAD and GIS connection. Water CAD V8i offers several advantages over other software's, including simpler model building using geographic modules and tools such as Load Builder and TRex, water quality modeling, fire flow analysis, optimization, and scenario management. Water CAD V8i is a user-friendly, adaptable, and high-quality modeling software suitable for a wide range of applications.

WATSYS (2014) is a water distribution simulation and modeling software that utilize Geographical Information Systems (GIS). WATSYS is effective for developing new water distribution systems and upgrading current networks. WATSYS analyzes water quality and distribution scenarios based on the EPANET software. WATSYS integrates seamlessly with AutoCAD, allowing for the creation of hydraulic networks using CAD drawings. WATSYS offers color-coded designs for visualizing pressure, altitudes, and flows at different nodes, fast data importing and exporting, and a built-in pipe inventory. WATSYS effectively simulates and models various distribution networks.

Water GEMS V8i (2014) is a hydraulic modeling software package with improved interoperability, network optimization, model development, and support for geospatial and asset

management systems. Sonaje and Joshi describe this software as very efficient and dynamic, offering comprehensive solutions for fire-flow analysis, water quality modeling, energy and capital cost management, and more. Many of the features and functions are common in Water CAD V8i and Water GEMS V8i which are streamlined model building, integration with the GIS and AutoCAD functionalities, optimized model calibration, design and its operations. Sonaje and Joshi cite the best part in the Water GEMS is the presentation of obtained results which is very attractive and appealing and can be presented with variety of graphical tools include ArcMap visualization, thematic mapping, contouring, profiling with color coding and symbology. With the ever increasing number of users Water GEMS has proved that ‘‘Water GEMS’’ is one of the most popular and user friendly hydraulic modeling and optimization software package. Water GEMS has strong design algorithm to meet the criteria of accuracy in design of water distribution networks, control of distribution network variables like flow, pressure, and velocity along with their optimization.

2.6.2.2.1 Modeling a System Using Water GEMS

Water GEMS is hydraulic simulation software offered by Bentley Systems. Bentley defines the parameters required for each model component, including:

- ◁ Reservoir: Reservoir details elevation, location, diameter, etc.
- ◁ Tank: Tank Parameters include base elevation, minimum and maximum levels, and tank diameter.
- ◁ Pump: The most important parameter defining the pump operation is the pump curve, elevation of the pump, curve of the pump and its location.
- ◁ Pipes: Diameter, length, and friction coefficient factors. Water GEMS assumes the pipe material has a Hazen William friction coefficient.
- ◁ Nodes: Elevations and Base Demands

Bentley Water GEMS states that successful simulations provide solutions for pressure, flows, pipe velocities, tank levels, pump cycles, water age, and constituent concentrations. Bentley describes the water GEMS software's capacity to analyze systems for both steady-state and extended period simulation.

Types of Simulations

(Walski, 1983); (Mohd et al., 2017) define simulation as the use of a mathematical representation or real-world system known as a model. A model can execute two sorts of simulations based on its purpose (observation or prediction). These include steady state simulation and extended period simulation (EPS) (Mohd et al., 2017).

Steady-State Simulation - The word "steady-state" refers to a system that remains stable over time, indicating equilibrium. The simulation's boundary conditions include constant tank and reservoir levels, hydraulic demands, and pump/valve operation. A steady-state simulation determines the network's equilibrium flows, pressures, and other variables under specific hydraulic demands and boundary conditions.

Real water distribution networks are rarely in a real constant state. Thus, the concept of steady state is a mathematical construct. Pumps are frequently turned on and off due to fluctuating demands and tank water levels. A steady-state hydraulic model resembles a hazy snapshot of a moving item rather than a clear photo of a fixed one. The mathematical concept of steady state can be a valuable tool for designers as it allows them to predict the response to certain hydraulic situations, such as peak hour demands or fires at specific nodes.

Steady-state simulations serve as the foundation for more complex simulations. Mastering the steady-state notion allows for better understanding of advanced topics including extended-period simulation, water quality analysis, and fire protection studies. Steady-state models are used to examine worst-case scenarios including peak demand, fire protection, and system component failures, where time is not a relevant factor.

Extended-Period Simulation – Determines the quasi-dynamic behavior of a system over a period of time, computing the state of the system as a series of steady-state simulations in which hydraulic demands and boundary conditions do change with respect to time.

Extended-period simulations require a large number of inputs. Before dealing with extended period simulations, it's important to test a model under steady-state conditions due to the large amount of data and potential actions during calibration, analysis, and design processes. Achieving satisfactory steady-state performance makes it easy to transition to EPSs.

Like all mathematical models, WDS model parameters require calibration before useful results can be obtained from simulation.

2.6.2.2.2 Calibration of Water Distribution Networks

There have been numerous definitions offered for the calibration of water distribution networks. According to Cesario and Davis (1984), calibration is the process of fine-tuning a model so that it accurately simulates field conditions. (Shamir & Howard, 1977) presented calibration as a process that includes both modeling and technical applications: (i) A modeling problem is the determination of the physical characteristics (basically, network configuration) and (ii) operational characteristics of an existing system, whereas engineering applications are the determination of the data that, when entered into a computer model, will produce realistic results. (Walski, 1983) argued that a water distribution network model is considered calibrated if it can predict flows and pressures with reasonable agreement to observed values. (Muzadzi, 2013) underlined that the calibration process should verify that the hydraulic model accurately predicts the network's behavior. To summarize, Calibration is a process of fine tuning a model simply adjusting roughness value of pipes and nodal demands in the order that the difference between field measurements (pressures and flows) and the simulated model results are minimized (Muzadzi, 2013). Guidelines for WDS model calibration have recently been established (Walski, 1983) but challenges still remain between theory and real world applications.

Bentley Water GEMS has a tool which is an additional module for the hydraulic modelling software such as Water CAD and Water GEMS Called “Darwin Calibrator” tool, It is used to calibrate water distribution networks.

According to Bentley Water GEMS, Darwin Calibrator allows the user to calibrate a model either manually or, with efficient genetic algorithms, in a more automated fashion. It allows for multiple calibration candidates to be presented so the best possible solution to a given system can be found. Solutions can also be exported into a new scenario for use in an existing water system. (Bentley) describe how the Darwin Calibrator can solve for three different fitness functions namely: (1) minimizing square differences, (2) minimizing absolute differences and (3) minimizing maximum absolute differences.

In 2001, Darwin Calibrator was used to calibrate water distribution model of the city Guayaquil. Guayaquil has a population of 2.3 million people located in Ecuador. The undertaker company that is responsible for operating and managing Guayaquil's water systems has adopted Water GEMS and Darwin Calibrator for hydraulic network simulation. (Muzadzi, 2013), a Darwin calibrator was used to calibrate the N8.3 pressure zone model of the Ankara water distribution system. The Darwin calibrator allowed engineers to identify and quantify unaccounted-for water while saving many trial-and-error hours (Wu et al., 2004).

The network hydraulic model has been widely used in practice and research institutions to predict leak size and location. It has been well developed and applied to water distribution system analysis in the last three decades (AWWA 2005). For leakage management, the network hydraulic model can be used for many purposes, including network zoning (Awad et al. 2009), leakage modeling as pressure dependent demand (PDD) (Almandoz et al. 2005); (Germanopoulos 1985); (Wu et al. 2010), leakage detection using Bentley Water GEMS and pressure management planning for leakage control (Burrows et al. 2003); (Tabesh et al. 2009); (Ulanicki et al. 2000).

2.6.2.2.3 Modeling Leakage

Modelling leakage depends on understanding the hydraulics of leaks and how to incorporate the hydraulics into existing models of the water distribution system. Hydraulic modeling of a network involves two approaches: Demand Driven Analysis (DDA) and Pressure Dependent Analysis (PDA). A classical Approach for hydraulic modeling/ simulation of hydraulic mode is called Demand Driven Analysis (DDA). DDA Assume nodal flows are always satisfied at demand nodes, regardless of available pressures at the Demand Nodes; it Assumes that the Demands are Independents of Pressure in the system; pressure has no influence on demand allocation. With DDA demand-pressure relationship is ignored and demands are always satisfied even when the nodal pressure is zero (Seyoum et al., 2011). DDA is only applicable when WDSs are simulated under normal conditions with adequate pressures. However, this is not always the case, for during mains failure or irregular water supply systems. Leakage is commonly included in nodal demands during WDS design, however this is unrealistic. Leakage, a kind of Pressure Dependent Demand (PDD), must be explicitly included when simulating hydraulic features. Researchers developed approaches to accurately simulate leaks in water distribution systems (WDS). Germanopoulous (1985) was the first to describe the inclusion of leakage terms in WDS models.

He applied empirical functions to relate users demand and leakage to network pressures and included the functions in the mathematical formulation of the network analysis problem (H. E. Mutikanga, Vairavamoorthy, et al., 2011).

Water GEMS now allows for the modeling of leakage as orifice flow using emitter hydraulics. A hydraulic model allocates leakage along a pipe to related nodes. The emitter nodes enable leakage to be modeled using appropriate pressure-dependent outflow relationships as shown in Equation

$$Q = k \times P^n \dots\dots\dots(2.3)$$

Where, Q is the leakage flow; k is emitter coefficient; P is pressure; n is leakage exponent (default 0.5 for orifice).

The leakage exponent n is the outstanding factor influencing the flow rate from a leak due to its position as exponent in Equation 2.3. Field studies on pressure-leakage analysis have shown that n typically varies between 0.5 and 2.79 and averages 1.0 (Farley & Trow, 2015) & (Thornton & Lambert, 2005) This means that leakage in water distribution systems is even more sensitive to pressure than traditionally assumed. Several factors have a significant impact on the range of leakage exponents:

- Ø The size and shape of the leak orifice, which depends on the pipe material and the type of failure (longitudinal or circumferential cracks, round holes, etc.).
- Ø The leak's ability to expand with rising pressure, which depends on the pipe material and the shape of the leak (round holes expand less than longitudinal cracks as pressure rises).
- Ø The surrounding soil.
- Ø The flow conditions at the leak orifice (laminar, transient or turbulent flow), which depends on the Reynolds number and the shape of the orifice.

However, the last two decades have seen tremendous improvements in network hydraulic modeling techniques. Several researchers have assessed leakage using network hydraulic simulations that fully incorporate leakage as PDD (Almandoz et al. 2005; Burrows et al. 2003; Giustolisi et al. 2008; Tabesh et al. 2009; Tucciarelli et al. 1999). Other than Tabesh et al. (2009) and Burrows et al. (2003) who evaluated their methodologies on real case studies in Iran and UK, the other methodologies have not been demonstrated on real WDSs to evaluate their effectiveness.

2.6.3 Leakage Detection and Location

Leakage detection is the main criteria in the water loss management. The leaks in water network systems are a growing concern for the water industry. Leak detection is the narrowing down of a leak to a section of a pipe network, whereas leak location refers to pinpointing the exact position of a leak (Pilcher et al. 2007). Leak detection surveys determine the exact location of leaks mainly by using acoustic equipment such as listening devices, noise loggers, and leak noise correlators (Hartley 2009). Acoustic equipment depends upon the noise (sound or vibration) generated by water leaking from a pressurized main. Conventional acoustic equipment can be unreliable for quiet leaks in non-metallic pipes and large-diameter pipes. Their effectiveness heavily relies on the experience of the user, and the process is time consuming. Recent advancements in technology and communication facilities have led to modern acoustic equipment that is more efficient and less dependent on user experience (Clark 2012; Hamilton 2012). Sensors equipped with multi parameter measurements (flow, pressure, and noise) are now available for network monitoring and leak localization (Koelbl et al. 2009a). Multi parameter measurement is a promising methodology that is likely to emerge as an alternative to DMAs for leakage monitoring (Farley 2012). Tethered in-pipe inspection and wireless technologies using video cameras, microphones, acoustic sensors, and smart balls have made it possible to find leaks in large-diameter pipes (Ong and Rodil 2012); (Stringer et al. 2007); (Wu et al. 2011). Leaks in water distribution networks could also be located with non-acoustic techniques such as tracer gas, infrared imaging, and ground penetrating radar (Fanner et al. 2007).

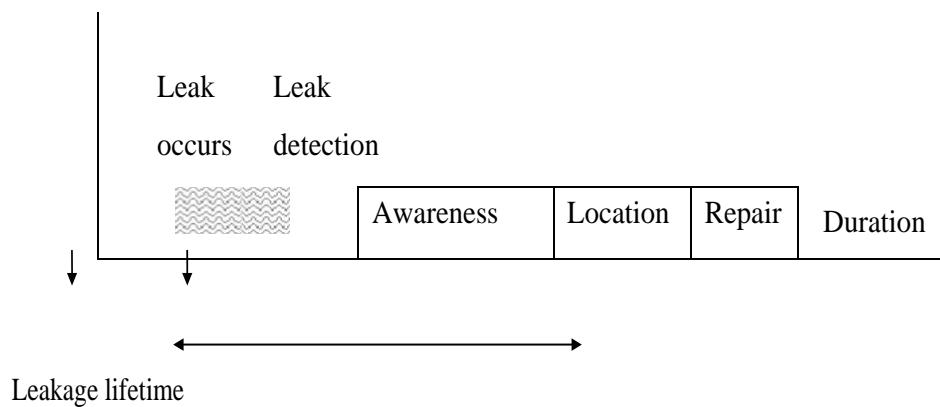


Figure 5 Life cycle of Leak

On the other hand, leakage detection can also be performed using the Darwin Calibrator in Bentley Water GEMS. By using the Darwin Calibrator Tool, you can predict the location and size of the water loss. The Darwin Calibrator can perform an optimized run to detect possible leakage automatically. It adjusts the emitter coefficient (a pressure-dependent demand) at nodes in order to get the model to match the field data.

2.6.3.1 Leakage Detection

The leaks in water network systems are a growing concern for the water industry. Without a calibration tool, leakage detection involves a series of steps that need to be performed to identify where leaks are located. In water GEMS, you can use the Darwin Calibrator tool to perform a leakage detection study. The Darwin Calibrator can perform an optimized run to detect possible leakage automatically. It adjusts the emitter coefficient (a pressure-dependent demand) at nodes in order to get the model to match the field data.

2.6.4 Leakage control

Leakage control involves using various methods and techniques to reduce leakage in water distribution systems. There are various reactive and active leakage control (ALC) techniques that include pressure management, mains rehabilitation, and speed repair of known leaks and bursts. Mains rehabilitation is successful in reducing leakage, but it is costly and difficult to justify just on leakage reduction. Speed in repairing mains failure is an inevitable reactive approach once failure has occurred. Pressure management is the most cost-effective and proactive way to reduce background leaks after laying pipes (H. E. Mutikanga, Vairavamoorthy, et al., 2011).

2.7 Pressure and Leakage Relationship

Effective pressure management in water distribution systems is crucial for leakage control (Environmental Protection Authority, 2007) & (Thornton et al. 2008). Understanding the pressure-leakage relationship is critical. The effect of operating at different pressures is modelled by FAVAD principles (May 1994) and FAVAD modified leakage equations (Cassa et al. 2010). In practice, the basic FAVAD equation for analyzing and predicting changes in leak flow rate (L0 to L1) as average pressure changes from P0 to P1 is (Lambert and Fantozzi 2010):

$$L1/L0 = (p1/p0)^{N1} \dots\dots\dots(2.4)$$

As the average service pressure grows to the power $N1$, the leakage ratio also increases. The dependability of forecasts depends on the ratio of average pressures and the anticipated $N1$ exponent. This is important for analyzing pressure control solutions to reduce leakage. Reducing pressure by half results in a 29%, 50%, 65%, and 82% reduction in leakage for $N1$ values of 0.5, 1, 1.5, and 2.5. Accurate measurement of pressure and $N1$ readings is essential. Experimental studies indicate that $N1$ values are near to 0.5 for leaks with small round holes in plastic and metallic pipes, and $N1$ values close to 1.5 for small leaks (undetectable background leakage) from pressure-sensitive joints and fittings (Environmental Protection Authority, 2007).

The pipe material significantly influences the pressure-leakage relationship. Water pressure generates tension in pipe walls. Material properties, such as modulus of elasticity, affect how pipes burst and respond to pressure changes. Increasing the internal pipe pressure produces two effects: fissures and small cracks that do not leak at low pressure may open and start leaking once pressure rises. Consequently, background leakage from small leaks can be significantly influenced by changes in system pressure. Furthermore, the area of existing leaks may increase and result in a rise in the leakage flow rate. This is also dependent on the shape of the leak and its ability to expand with rising pressure (Fallis et al., 2011).

2.8 Apparent losses in Water Distribution System

Apparent Losses (AL) often referred to as commercial losses. Apparent losses are the non-physical losses in that no water is physically lost from the distribution system. They relate to water that is being consumed but not being paid for. Apparent losses include all types of inaccuracies associated with customer metering (worn meters as well as improperly sized meters or wrong type of meter for the water usage profile) as well as systematic data handling errors (meter reading, billing, archiving and reporting), plus unauthorized consumption (theft or illegal use) (AWWA, 2013). According to a World Bank report, approximately 16 billion m^3 of water every year is apparent loss, causing utilities worldwide to lose revenue estimated at US\$6.5 billion every year. The Asian Development Bank (ADB) estimates that 50–65% of NRW in Asian water utilities is due to apparent losses (Mcintosh, 2003).

2.8.1 Apparent Losses in Developing and Developed Countries

2.8.1.1 Apparent Losses in Developed Countries

Apparent losses result in appreciable revenue loss for water utilities and distort the integrity of consumption data required for various management decisions and engineering studies. Apparent losses are perceived to be a problem for developing countries, significant figures of apparent losses in the developed countries have been reported, particularly for systems with universal customer metering. For example, the 2006 fiscal year water audit for the city of Philadelphia (USA) estimates apparent losses at 21 million m³ (AWWA, 2013). Although the city's real losses are almost four times its apparent losses based on volume, in financial terms, apparent losses were estimated at US\$20 million compared to US\$4 million for real losses. This is because apparent losses are valued at the retail price charged to customers, whereas real losses are valued at the variable production cost (AWWA, 2013). In England and Wales, with 37% household metering, apparent losses are estimated at 118 million m³/year, with illegal use estimated at 32 million m³/year and meter under-registration at 86 million m³/year (OFWAT 2010). Many water utilities are increasingly migrating from traditional manual meter reading to automated meter reading and advanced metering infrastructure (AMI) as a way of minimizing apparent losses due to meter reading and data handling errors (AWWA, 2013). AMI systems with smart meters provide additional advantages of post meter leak detection and management.

2.8.2.2 Apparent Losses in Developing Countries

This problem is more pronounced in the water utilities of the developing countries. According to (Kingdom et al., 2006), about 30 000 ML/ day of water is delivered to customers in the developing countries, but is not paid for because of water theft, employee's corruption and poor metering practices. They estimate that about US\$3 billion/year of revenue is lost in the developing countries due to commercial losses. They go on to conclude that the financial viability of utilities in developing countries is constrained as a result and this hampers necessary service expansions especially for the poor.

The Apparent losses in some developing countries have been reported in various identified studies and are summarized in (Table 2.3) was given by (H. E. Mutikanga, 2012)

Table 4 Apparent Losses in some developing countries

Work	Country	City/Utility	NRW(% of SIV)	AL (% of NRW)	UU (% of AL)	MI (% of AL)
Schouten & Halim (2010)	Indonesia	Jarkata/Palyja	48	39	53	47
Sharma & Nhemafuki (2009)	Nepal	Bhaktapur	54	56	17	83
		Dhulikhel	16	25	80	20
Dimaano & Jamora (2010)	Philippines	Manila/Maynilad	65	20	69	31
Shin et al (2005)	South Korea	Busan	26	14	0	100
Makara (2009)	Papua New Guinea	Eda Ranu/Port Mores by	64	16	52	47
Garzon-Contreras & Palacio-Sierra (2007) Garzon- Contreras et al (2009)	Colombia	Medllin/EPM	33	32	75	25
		Cali	44	48	56	44
		Bogota	37	43	55	45
Garzon – Contreras et al (2005)	Colombia	40 Water Utility	41	34	ND	ND
Medeiros et al (2010); Batista & Mendonca Jr.(2009)	Brazil	Sao Paulo SABESP	ND	33	26.6% **	20.24%*
Mutikanga et al (2009b,2011)	Uganda	Kampala/NW SC	40	52	43	57(22*)
Lievers & Berendregt (2009)	Ghana	Accra - Tema	55	55	18%***	ND
Seago et al (2004); Wegelin et al (2011)	South Africa	All Municipality	23	5 to 50	30	30
Bidgoli (2009)	Iran	Tehran/NWW EC	36	44	26	67

From Table above, the following summary was made

- Ø NRW expressed as a percentage of system input volume (SIV) varies from 16% to 65% with an average of 41%. In terms of volume, the highest NRW is 1,497 ML/d in Maynilad, Manila and the smallest is 61 ML/year in Dhulikhel, Nepal.

- Ø AL ranges from 14 to 56% of NRW with an average of 34%
- Ø The most significant component of AL is metering inaccuracies (MI) with an average of 54% of AL but ranges from 20% to 100% where illegal use of water has been reported to be zero
- Ø Unauthorized Use (UU) averages 46% of AL with a range of 0 to 80%. Maynilad in Manila has the highest figure in terms of volume lost (202 ML/d). This is more than twice the amount of estimated illegal use for England and Wales.
- Ø In Kampala city, unauthorized consumption occurred in many different ways and was categorized in a chronological order depending on their magnitude as follows: (i) illegal reconnections after disconnection for non-payment of bills (40%); (ii) consumption meter bypass (35%); (iii) illegal connections (16%); and (iv) meter tampering and reversing (9%) (H. E. Mutikanga, 2012).

However, In Addis Abeba Water Distribution System Apparent Losses generally not very well understood, little work was done, also there is no clear estimation.

Apparent Losses include all types of inaccuracies associated with customer metering, data handling errors (meter reading and billing) and unauthorized consumption (theft or illegal use).

2.8.2 Metering inaccuracies

Water meters are imperfect tools, much like any other measurement tool. Every water meter, regardless of technology, has specific measuring constraints that prevent it from recording the exact amount of water used by customers when it is installed. (Criminisi et al., 2009) & (H. E. Mutikanga, Sharma, et al., 2011a). The user is frequently not paid for water that is consumed because it is not recorded. The meter is said to be under-registering or exhibiting negative error in such circumstances. Contrarily, some circumstances may have the reverse effect, causing the meters to report greater water consumption than was actually used, depending on the meter technology. After that, it is said that the meter is over registering or displaying positive error. (Alegre et al., 2016). In either instance, it's crucial to quantify the magnitude of these measuring errors because meter inaccuracies are recognized as a significant contributor.

The largest contributor to apparent losses has been found to be metering errors (Rizzo & Cilia 2005) and has therefore received greater attention in literature. Examples of relevant work

quantifying metering errors include (Noss et al., 1987), (Yee, 1999) and (H. E. Mutikanga, Sharma, et al., 2011a). This is because the vast majority of meters employed by water utilities are mechanical meters which lose accuracy with age due to wear and tear of the mechanical components. The majority of these studies involve the determination of the weighted meter accuracy using the methodology outlined in (Noss et al., 1987) , (Yee, 1999) and (F. . Arregui et al., 2006) that involves the determination of metrological performance of a meter at different flow rates and the user demand temporal pattern (Criminisi et al., 2009) . Such methodology can be very involving and costly for water utilities and as such, more cost effective alternatives are more ideal. (Mbabazi et al., 2015) and (Couvelis and van Zyl., 2012) employ such alternatives through the use of a technique described by (F. Arregui et al., 2003) of comparative billing analysis as a means to support decision making in water meter management.

2.8.3 Unauthorized Consumption

Unauthorized water use occurs through deliberate actions of customers or other users who draw water from the system without paying for it. It occurs in many ways including illegal connections, illegal-reconnections, meter by-pass, meter tampering and abuse of fire hydrants. Unauthorized water use is a socio-technical problem that requires not only engineering solutions but also socio-cultural approaches that require changes in community behavior and attitudes toward water use as well as a strong stance against fraudulent practices of utility staff and water users. The socio-cultural approaches including working with local communities at the lowest administrative and street levels (territory management concept) have been reported to have been major drivers in reducing NRW in some Asian cities such as the east zone of Metro Manila where NRW has been reduced from 63% to 11% in the past 14 years saving over 0.6 million m³ of water per day (Luczon and Ramos 2012). Pressure management data from WDSs can be used in algorithms for inverse calculations to detect unauthorized use (Liggett, 1995). In a recent laboratory study, it has been shown that location and characteristics of illegal branches can be detected by means of fast transient tests (Meniconi et al., 2011).

2.8.4 Systematic Data Handling error

Meter reading and data handling errors arise during the process of meter readings (gathered manually or automatically), data transfer to the billing system and archiving of customer

consumption data. These errors can be caused by picking wrong meter readings (intentionally or accidentally), failure of Automated Meter Reading (AMR) equipment, wrongly captured data by billing assistants, erroneous system volume estimations and other policy and billing adjustment shortcomings. Leveraging the well-developed IT, metering technologies, and billing procedures and policies will be imperative for minimizing these errors. Many water utilities are increasingly migrating from traditional manual meter reading to Advanced Metering Infrastructure (AMI) as a way of minimizing apparent losses due to meter reading and data handling errors (AWWA, 2013). AMI systems with smart meters provide additional advantages of post-meter leak detection and management.

Assessing apparent water losses

In the absence of adequate data and proper methodology, most developed countries use default values or rules of thumb (e.g. AL is computed as 1% to 3% of total system input volume in Australia) which tend to be lowest values for well managed water systems, for component computation of apparent losses (Lambert A.O., 2002). However, these default values may not be appropriate for developing countries where illegal use of water is rampant and meter management policies are ineffective. In a model (BENCHLEAK) developed for leakage management by the South African Water Research Commission, McKenzie et al. (2002) applied a default value of 20% of total water losses. As rightly acknowledged by the authors, this approach was too simplistic, unrealistic and not scientific. For example, in Johannesburg city, an upscale area such as Sandton is likely not to have the same level of illegal use as Soweto township and to assume a default value of 20% for AL is grossly erroneous. (Seago et al., 2004) proposed a simplified approach for assessing apparent losses for South African water utilities based on age of water meters; water quality and qualitative information (very low to very high and poor to good) provided by utilities.

2.9 The causes through which water is lost by water utilities

2.9.1 Causes of Real losses

Pipes and fittings of different dimension, material and age are installed below ground, where they are subject to a multiplicity of factors which cannot be registered and controlled on a regular basis. Additionally, Lambert and McKenzie specify four key system-specific factors for real water losses, namely the length of the mains, the number of service connections, the location of the

customer's meter and the average operating pressure in the system (when the system is pressurized). These factors vary from system to system (Charalambous et al., 2002)

The multitude of active and passive interactions between pipes and their environment frequently lead to damage and leaks. In simple terms, the assets of water distribution networks comprise (a) pipes and joints, (b) valves and fittings and (c) storage tanks and pumps. The causes and factors influencing water losses thus can be classified according to the type of asset (Fallis et al., 2011).

(a) Defective pipes and joints

Material, condition and age of the pipe

Material: besides material failures caused during manufacturing (insufficient wall thickness, absence of corrosion protection on steel pipes, poor reinforcement covers on concrete pipes), damage is also brought about by inappropriate use of certain materials: soft water (especially untreated water from dams) with a high carbonic acid (CO₂) content and low calcium concentrations, or with high sulphate concentrations are known to affect concrete aggressively. Reinforced concrete pipes as well as the internal and external cement mortar lining of cast iron pipes are affected.

Condition: all metallic pipes are exposed to physical and electro-chemical corrosion. Corrosion will reduce the wall thickness and decrease the pipe's ability to withstand water pressure and external stress. The most common causes of corrosion are aggressive water and soil as well as stray current.

Age: many factors influencing leakage are age-dependent. Consequently, the age of a pipe section can appear to be the most significant factor for leakage. Nevertheless, age is not necessarily a factor, if the pipe has been carefully designed and installed, maintenance is carried out at regular intervals and external conditions are favorable.

Design and installation, workmanship

Design: mistakes made during the planning phase may influence leakage from pipes, such as the incorrect material choice, insufficient dimensioning for the actual pressure, inadequate corrosion protection measures or incorrect alignment (e.g. along slopes prone to landslides or adjacent to tree roots).

Storage and laying of pipes: improper storage may damage pipes even before installation. Grass roots can penetrate the bituminous coating of steel pipes. Dragging PE pipes over concrete surfaces or stones will cause chamfers which are more susceptible to future leakage. Extended sunlight exposure causes PE pipes to become brittle. Heavy machinery used for transport and installation may also cause damage.

Bedding: the selection of inappropriate material for pipe bedding is a frequent cause of damage: Coarse or rocky bedding material spoil external coatings on steel or cast iron pipes and support corrosion. Longitudinal and spiral cracks may appear in PE and PVC pipes as a result of stony bedding materials. Insufficient backfill and compaction of the pipe trench may cause subsidence. These uncontrolled soil movements can also trigger socket disconnection or pipe ruptures.

Joints: unprofessional execution of pipe joints is a further reason for leakage. Welded steel pipes often lack proper internal and external corrosion protection along the weld seam. If unskilled or poorly trained welders execute the relatively new technique of welding PE pipes, it frequently contains defects due to insufficient heating and pressing of the pipe ends. Leakage can also occur if socket pipes exceed the maximum permitted angular bending or if water hammer and high pressures affect joints that are not force locked.

Pressure

High pressure: increasing pressure will result in a higher flow rate from existing leaks and an augmented occurrence of new pipe bursts and leaks. As pressure mounts, leakage rates rise to a much larger extent than would be predicted by the theoretical square root relationship between pressure and leakage rates. Aged valves and fittings may lack sufficiently robust dimensioning for high pressures.

Poor pressure: poor pressure may complicate leak detection efforts because the water is less likely to reach the surface. Furthermore, decreased noise levels from the leak impede acoustic leak location methods and may cause longer leak runtimes.

Pressure variations: marked variations in pressure within the system may lead to material fatigue and thus to leakage, mainly in plastic pipes.

pressure surges: pressure surges (water hammer) mainly arise from inappropriate control mechanisms and can cause pipe fractures, disconnect joints and damage valves and fittings, hence leading to leakage

Soil and groundwater

Soil type: the prevailing soil type has significant effect on the runtime of leaks. While water escaping into cohesive soils (e.g. clay or silt) may soon appear at the surface, leaks into non-cohesive soils (e.g. sand or gravel) tend to drain away below ground, making leaks more difficult to detect.

Soil aggressiveness: most non-cohesive soils are not aggressive. Cohesive soils may negatively affect the external corrosion of metallic pipes due to differing levels of dissolved salts, oxygen, moisture, pH and bacterial activity.

Soil movement: soil movement is caused by changes in the temperature and moisture content (changing groundwater levels cause cohesive soils to contract or expand), heavy frost as well as subsidence prompted by incorrect pipe bedding, mining activities or earthquakes. Construction work, increased surface loads or pipe repair works may also trigger soil movement. Soil movement may cause pipe joints to disconnect and pipes to rupture.

Traffic

Traffic load: many pipes that were originally laid underneath pavements are now situated under the roads as traffic and road widths have increased. More vehicles and the high axle loading of modern trucks place an additional burden on the pipes.

Pavement: the water outlet may appear far away from the damage due to concrete and tarmac pavements, thus hampering leak location efforts.

Stray current: 1 ampere erodes approximately 10 kg of iron each year. Stray currents from DC-powered railway systems (trams) and reinforced concrete foundations thus aggravate the external corrosion of metallic pipes.

Third-party influence

The absence of infrastructure documentation (e.g. as-built drawings) or improper execution of construction work may damage pipes, either directly through excavators or indirectly through vibrating construction machines or heavy vehicles. Damage may be detected instantly or after a delay, making the cause harder to detect.

Other factors

There are numerous other factors, e.g. the number of valves and service connections (known as the weak points within the distribution system) per kilometer of pipe, the length and diameter of

the mains, the average length of service connections, the depth at which the pipes were laid as well as background leakage from joints, fittings and valves. The operational management of the system also influences leakage, e.g. in terms of water from different sources being mixed within the pipe system (risk of corrosion), the percentage of time the system is pressurised and the inspection, maintenance and leak detection strategies that influence the runtime of detected and undetected leaks. (Alegre H., Hirnir W., Baptista J., 2006)

(b) Defective valves and fittings

Leaks from valves and fittings include breakage, deformations or material failures at the valve body as well as leaking gaskets at joints, bonnets or stems. Rough handling or an absence of maintenance often causes these leaks. Defective valves and fittings may contribute significantly to real water losses in spite of rather low leak rates. These deficits often remain undiscovered for a long time without a regular servicing programme. Considerable water losses may also occur from defective fire hydrants and public water standpipes.

(c) Defective storage tanks and pumps

Water losses from storage tanks are caused both by structural damage and by operational failures, such as faulty or lacking system controls which may result in overflow. Structural damage involves cracks, holes or delamination at tank walls or floors, leakage due to inferior concrete quality, as well as sealings and pipe penetrations that are not watertight. Water losses from tanks are often underestimated and, though easy to detect, repairs are usually elaborate and expensive. Water losses from pumps are usually caused by defective pump shaft seals. The amount of leakage is negligible in most cases, but flooding pump chambers and electric equipment is a nuisance and should be avoided by undertaking proper maintenance.

2.9.2 Causes of apparent loss

Apparent losses are not physical but rather financial losses. They are caused by unauthorized consumption (theft and illegal use) and metering inaccuracies (Lambert A.O., 2002). While unauthorized consumption causes are directly related to water utility management and may be reduced by improving company procedures, water meter inaccuracies are considered to be the most significant and hardest to quantify. (Criminisi et al., 2009).

Metering inaccuracies

Meters were installed horizontally in order to minimize any effect caused by orientation. (F. J. Arregui et al., 2016). The most common cause of meter failure is related to water quality, as shown in the study of (H. E. Mutikanga, Sharma, et al., 2011b) Due to high frequency of bursts and leaks coupled with poor pipe repair practices and inadequate mains flushing, a lot of silt and other suspended particulates enter the water system and get lodged in the meter drive mechanisms (reducing gear trains and oscillating piston) halting them from moving. In addition, a system with corroded pipes operating under intermittent conditions is likely to re-suspend a lot of deposited particulates in the pipe, which compounds the meter blockage problems.

In a water distribution system, metering errors are the main cause of apparent losses (Rizzo & Cilia, 2005). Water meters can become inefficient for a number of reasons, such as meter wear and tear, incorrect installation practice, a lack of maintenance or calibration, the use of the incorrect meter type and class for the application, incorrect meter sizing, and problem with demand profiles or demand types. assuming that the meter selection, sizing, installation, and calibration have been performed correctly, apparent losses brought on by water meter aging and demand profile still persist.

There are many different kinds of water meters, including volumetric, electromagnetic, and ultrasonic ones. Volumetric, single-jet, and multi-jet versions are the most often used residential revenue meters. This sort of meter frequently under registers due to aging or excessive wear and tear on its working parts. Poor water physical and chemical qualities, environmental issues, and entrapped air bubbles creating high velocities are the most common causes of wear and tear (Rizzo, 2006).

(F. Arregui & Jr, 2005) presented real field and laboratory data on the impact of several parameters on the accuracy of both domestic and industrial water meters and on different meter technologies (single-jet, multi-jet, oscillating-piston, Woltman and Tangential meters). Incorrect mounting position, wear of moving parts, suspended solids and deposits, leaks and user's storage tanks, and partial blockage of the inlet strainer are all said to influence the error curve of domestic water meters (although this varies with the meter technology). In addition to these factors, velocity profile distortions and proper meter sizing also have an effect on water meter error for industrial applications.

Water meter inaccuracy of the customer meters are commonly attributes to apparent losses. The inaccuracy of the consumer meter could be due to degradation of its internal mechanical parts, build-up of particles inside the measuring chamber, poor quality meters and incorrect installation of the meters in field. This problem could generate significant meter under-registration which could lead to loss of utility’s revenue.

The following are major influence elements that affect the accuracy both for domestic and industrial water meters based on their technology.

Mounting position

An incorrect mounting position of the water meter increases the friction of the moving parts. which varies depending on the design of the impeller bearings used in each meter, the error at low flows becomes more negative even though for flows higher than the transitional flowrate the accuracy is maintained (F. Arregui & Jr, 2005). The study for a water utility in one of the islands of Indonesia identified that the degree of inclination or tilting position of the melti jet water meter is directly proportional to the increase of negative meter errors, using the comparison of average meter errors at different flow rates and the variances of errors according to the meter positions.

Meter Position	Horizontal [1]	45 degrees [2]	90 degrees [3]	Variance bet. [1] and [2]	Variance bet. [1] and [3]
30 liters/hour	-3.2%	-12.6%	-17.5%	9.4%	14.3%
120 liters/hour	3.25%	1.95%	0.77%	1.3%	2.48%
1500 liters/hour	1.66%	0.77%	-0.29%	0.89%	1.37%

Even the newly purchased meters and those meters with less than a year of service age will give negative error when they are incorrectly installed. This phenomenon can be related to famous quote; “age doesn’t matter” which technically means that whatever the age of meter when it is installed in non-horizontal manner this will under-register water consumption.

The study identified, the effect of incorrect meter installation could lead to Apparent loss of roughly around 20% and the rate wear and tear of the meter would be faster than normal due to the friction between the impeller and pivot spindle which will damage the measuring components in the long run

Another important consideration about the mounting position is that if it is not correct, according to the manufacturer recommendations, it may lead to a higher degradation rate of the meter. The reason is that in such cases the moving parts of the meter are not working and standing properly (F. Arregui & Jr, 2005).

Suspended solids and depositions

These two parameters have a substantial impact on water meters. Positive displacement meters may stop when a particle bigger than the spare space between the piston/disc and the chamber passes through the strainers of the meter. Although this element is constructed in such a way that this cannot happen in principle, actual usage demonstrates that it is conceivable.

The effect on velocity meters can be significantly different depending on its construction (F. Arregui & Jr, 2005). One of the reasons for such behavior is the adjusting device used in multi jet and some single jet water meters to bring the accuracy curve into the permissible error interval. This type of adjusting device derives some of the water through an alternative circuit, by-pass conduit, reducing the energy transfer at a given flowrate to the impeller.

Rate of flow

Water consumption at low flows is the most difficult to measure. Meters will generally lose accuracy at the low flow rate and it's at this low flow that the meter is more prone to not register at all (Armentrout, 2011) Under these working conditions the energy transfer from the fluid to the sensing element is very small and any increase in friction, caused by any factor, may stop the impeller or the piston at lower flowrates. Realize also that some meters will over-register just as easily as under-register. Some meters will under-register on low flows and over-register on higher flows. Furthermore, considering that the accuracy of a water meter decays at a faster rate at low flows, this under-registration will increase more rapidly than in other type of users (F. Arregui & Jr, 2005)

User's privet storage tanks

In countries suffering water shortages (such as those in arid and semi-arid climates), the use of private storage tanks (Rizzo & Cilia, 2005); (Criminisi et al., 2009) affects the share of the consumption that occurs at low flow rates. The use of storage tanks is a common practice for

coping with water scarcity. During periods of water shortage, a discontinuous water supply and water resource rationing are often the primary measures adopted by water utilities to distribute the limited water as efficiently as possible. Users compensate for this intermittent water service by collecting water in a tank during serviced periods and redistributing it when public water service is not available (Fontanazza et al., 2008).

Private tanks are filled using a proportional float valve that opens partially or totally as a function of tank water level and network pressure. While the users are receiving a continuous supply, the tank is usually full, and the float valve opens as soon as the tank's water level falls. During periods of high consumption, the tank's water level drops, the float valve opens 20 completely, and water enters the tank at a high flow rate. During periods of lower consumption, the water level does not fall as much, the valve opens only partially, and the flow rate passing through the meter and entering the tank is very low. The meter is thus forced to work in the lower part of its measuring range where error is very high. Furthermore, the slow closure of the float valve induces flows that are lower than the meter's starting flow and thus are not registered. When the users experience intermittent supply (de Marchis et al., 2010) & (De Marchis et al., 2011), water flows into the tank only when the network pressure at the user connection is sufficient to supply the tank. When the network pressure is low, tank water levels drop to meet user needs, with tanks often almost empty by the time the network pressure increases. As a consequence, the float valve is completely open and allows water to pass at a very high flow rate.

The effect of a tank on consumption flow rates raises the global error of a meter, ranging from approximately -10 % for a new meter to -40 % for a worn-out meter. In terms of apparent losses, experimental evidence shows that the average under-registration of worn-out meters ranges from approximately 10 to 50 % of total household consumption (Criminisi et al., 2009)

Partial blockage of the inlet strainer

Partial blockage of the inlet strainer may affect the accuracy of the meter. Some water meters are sensitive to the momentum of the water impacting on the impeller or the turbine. (F. Arregui & Jr, 2005)

2.10 Performance indicators in water loss management

2.10.1 Performance Assessment System (PAS)

Performance assessment systems examine management targets, strategies and overall effectiveness. Performance measurement is used by a variety of organizations, including regulators (for example, OFWAT in the UK), financial institutions (such as the World Bank), policymakers, and utility management. The task of measuring and evaluating performance is accomplished by well-defined performance indicators (PIs) (H. Mutikanga et al., 2010). The assessment of the undertaking's performance with the use of PIs can measure the quality of service and the utility's effectiveness and efficiency; make transparent the comparison between the objectives, provide benchmarking between similar undertakings and encourage them to provide an improved service (Alegre et al., 2016); (Alegre H., Hirnir W., Baptista J., 2006).

Conducting water audits allows a utility to measure its water loss performance over time and compare it to other utilities (Thornton, 2005). Benchmarking uses a set of performance indicators to numerically assess various components of the distribution system. Performance indicators must be consistent, repeatable, and given in defined units (EPA, 2010).

The AWWA/IWA water audit methodology has a standard array of performance 33 indicators that the public water system (PWS) can track annually when compiling the water audit. Performance assessment is crucial for comparing water utilities' performance against peers. Water utilities need to use performance assessment systems to improve service delivery and meet international requirements. A survey of 21 African water companies found that 12.9% run more efficiently than their contemporaries (Estache & Kouassi, 2002). These water utilities often operate under a weak governance and financial framework, with utility managers having to face multiple political and economic constraints (Kingdom et al., 2006) & (Estache & Kouassi, 2002). Thus, this finding supports the commonly held view that Africa's water sector operates at unacceptable levels of technical inefficiency (Estache & Kouassi, 2002). Water utilities have to provide some form of service to customers on a daily basis with mostly deteriorated infrastructure (Kingdom et al., 2006). This is evidenced by the Zimbabwean situation where despite the country having high

access to water services, its dependency on surface water stands at 7% of the total demand (Makaya & Hensel, 2014).

For benchmarking purposes, performance indicators can be classified as Operational, Asset serviceability, Meter management, Legal use management, Human Resources Management, and Economic and Financial. Ultimately, non-revenue water, a proxy of water losses, is a noble starting point in assessing water distribution systems efficiency (Eugine, 2015).

The most widely used indicators for WLM are those developed by IWA (Alegre et al., 2016); (Lambert et al., 1999) and adopted by the AWWA (AWWA 2003). These indicators are presented in Table 2.1. According to (Eugine, 2015); (H. Mutikanga et al., 2010); & (Kingdom et al., 2006) the percentage non-revenue water is the most commonly used performance indicator (PI) for assessing water losses and setting targets. NRW is commonly used to assess the efficiency of water supply systems. According to (Eugine, 2015), citizen satisfaction with municipal services is directly related to their actual performance compared to initial expectations. The fundamental problem of using percentage NRW as an indicator is that it is impacted by consumption patterns, not the utility's WLM (H. Mutikanga et al., 2010).

Table 5 IWA/AWWA performance indicators for water losses

Level	Water Resources	Operational	Financial
Basic	Inefficiency of use of water resources: Real Losses as a percentage of System input Volume	Water losses: volume/service conn/year)	NRW : NRW as a percentage of system input volume
Intermediary		Real losses: volume/service. Conn/day (when system is pressurized)	
Detailed		Apparent losses: volume/service. Conn/year	NRW : value of NRW as a percentage of the annual cost of running the water system.
	Infrastructure leakage index (ILI)		

Source: (AWWA, 2003)

The other most used technical PI is the infrastructure leakage index (ILI).

2.10.2 Infrastructure leakage Index (ILI)

The infrastructure leakage index (ILI) defined as the ratio of current annual real losses (CARL) to unavoidable annual real losses (UARL). The UARL represents the lowest technically achievable annual Real Losses for a properly maintained and managed (maintained, repaired, and rehabilitated) system. The ILI measure a distribution network's ability to manage real losses under current operating pressure. In well-managed WDSs, ILI is equal to or close to 1 and often rises with age. Table 2.2 presents the default values used in computing UARL (Lambert et al., 1999).

Table 6 Standard unit used for calculating UARL

Infrastructure Component	Background (undetectable) losses	Reported bursts	Unreported bursts	Total UARL
Mains	20 L/km/h*	0.124 bursts/km/year at 12 m ³ /h* for 3 days duration	0.006 bursts/km/year at 6 m ³ /h* for 50 days duration	18(L/km mains/day/m of pressure)
Service connections to edge of street	1.25 L/conn./h*	2.25/1000 conns./year at 1.6 m ³ /h* for 8 days duration	0.75/1000 conns./year at 1.6 m ³ /h* for 100 days duration	0.8 (L/conn/day/m of pressure)
Service connections from curb-stop to meter**	0.5 L/conn./h*	1.5/1000 conns./year at 1.6 m ³ /h* for 9 days duration	0.5/1000 conns./year at 1.6 m ³ /h* for 101 days duration	25 (L/km of service conn./day/m of pressure)

*All flow rates are specified at a reference pressure of 50 m. ** Assuming average length of service connection from curb-stop to customer meter. (Source: (Lambert et al., 1999))

The ILI indicators are provided as a rough guide and should not be taken as strict limits. However, in many parts of Africa and Asia, very high ILI values are often experienced and the limits given in Table 2.6 tend to become meaningless. The main purpose of the ILI indicator is to help identify areas where leakage/losses are abnormally high to ensure that action is taken in the most appropriate areas (Winarni, 2009).

The matrix in Table 2.6 shows the anticipated ILI levels and physical losses at different pressure levels.

Table 7 Physical loss target matrix (Source: (Eugine, 2015))

Technical Performance Category		ILI	Physical Losses[liters/connection/day] (When the system is pressured) at an average of				
			10m	20m	30m	40m	50m
Developed Countries	A	1-2		<50	<75	<100	<125
	B	2-4		50-100	75-150	100-200	125-250
	C	4-8		100-200	150-300	200-400	250-500
	D	>8		>200	>300	>400	>500
Developing Countries	A	1-2	<50	<100	<150	<200	<250
	B	2-4	50-100	100-200	150-300	200-400	250-500
	C	4-8	100-200	200-400	300-600	400-800	500-1000
	D	>8	>200	>400	>600	>800	>1000

To guide further network development and improvement, utility managers can use a matrix with the following categories (Farley & Trow, 2015):

- Ø Category A — Good. Further loss reduction may be uneconomic and careful analysis is needed to identify cost-effective improvements.
- Ø Category B — Potential for marked improvements. Consider pressure management, better active leakage control and better maintenance.
- Ø Category C — Poor. Tolerable only if water is plentiful and cheap, and even then intensifies NRW reduction efforts.
- Ø Category D — Bad. The utility is using resources inefficiently and NRW reduction programs are imperative.”

The IWA water loss task force team studied 197 water initiatives worldwide and found ILI figures ranging from 0.3 in Austria and the Netherlands to 598 in South East Asia (McKenzie et al. 2007). The researchers concluded that ILI is unsuitable for comparing systems in developed and developing countries with often intermittent supply and low pressure water distribution systems. The suggested ILI target values are 2 for well-managed systems in developed countries and 5 for developing countries.

However, ILI is still a valuable tool as an indicator of real losses in WDSs and where appropriate it should be applied in its current form or with appropriate modifications mainly focusing on improving assumptions made in estimating ILI components particularly the UARL (black box) to suit local conditions and advances made in technology over the years.

2.10.3 Apparent Loss Index (ALI)

Using the ILI analogy, the apparent loss index (ALI) has been proposed (Thornton et al., 2008); (Rizzo, 2006); (H. Mutikanga et al., 2010). The ALI is defined as the ratio of the current annual apparent losses (CAAL) to unavoidable annual apparent losses (UAAL). In absence of a reliable UAAL, a base value of 5% of water sales is recommended as a reference value.

However, the benchmark reference value of 5% of water sales is high for most water utilities in developing countries. High figures for AL as percentage of water sales (revenue water) have been reported in various studies (H. Mutikanga et al., 2010). Table 2.7 shows the variation of AL of water sales for different countries.

Table 8 Variation of AL of water sales for different countries *Source: (Eugine, 2015)*

City and Country	% AL	Data Source
Kampala, Uganda	37	(H. Mutikanga et al., 2010)
Lusaka, Zambia	33	(Sharma and Chinokoro 2009)
Manila, Philippines	16	(Dinaano and Jamora 2010)
Jakarta, Indonesia	36	(Schouten and Halim 2010)
Philadelphia, USA	9.6	(AWWA 2009)
England and Wales	2.8	(OFWAT 2010)

AL in systems with customer storage tanks cannot be directly compared to systems on direct mains pressure supply due to the ball-valve effect, which intensifies AL (Lambert, 2002). More relevant performance indicators and indices are needed to meet the unique characteristics of WDSs in developing countries and compare performance among utilities (H. Mutikanga et al., 2010). Table 9 shows ALI performance bands for both developed and developing countries.

Table 9 ALI performance bands Source: (Eugine, 2015)

Region	Technical performance group	ALI	Remark
Developed Countries	A	1-2	Acceptable performance. Further reduction may be uneconomical unless if the cost of water is very high.
	B	2-3	There is room for improvement.
	C	3-4	High revenue losses, acceptable where cost of water is very low.
	D	>4	Very inefficient with poor meter management practices and in adequate policies for revenue protection. Urgent action required to minimize revenue losses.
Developing Countries	A	1-2	Acceptable performance. Further reduction may be uneconomical unless if the cost of water is very high.
	B	2-4	There is room for improvement.
	C	4-6	High revenue losses, acceptable where cost of water is very low.
	D	>6	Very inefficient with poor meter management practices and in adequate policies for revenue protection. Urgent action required to minimise revenue losses.

Water utilities should prioritize auditable financial metrics over others due to their diversity. Prioritize auditable indicators over non-auditable ones. This is because revenue and revenue collection operations are crucial for the rational efficiency of water utilities (Eugine, 2015).

2.10.4 Benchmarking

Benchmarking is a "best practice" process approach that compares a system's efficiency in terms of quality and work processes to others (H. Mutikanga et al., 2010). Benchmarking is typically used to increase productivity and efficiency by learning from "best practices," as well as to manage system efficiency and quality development in comparison to other sectors. Benchmarking is commonly used to improve system integrity and optimize operations. However, Benchmarking alone cannot address all water supply concerns for utilities (Lambert, 2002).

According to (Eugine, 2015), one of the successes of benchmarking was reported on water loss management. In England and Wales, leakages were decreased from 5 million cubic meters per day in 1995 to 3 million cubic liters per day by 2010 (OFWAT, 2010). A decrease in water leakages accounts for around 35% of the SIV. The key causes of this milestone are technological innovation, enhanced regulatory frameworks, and continual performance improvement. Table 2.7 shows countries that reported on water loss management using benchmarking approaches.

Table 10 Water loss management using benchmarking approaches for different countries *source: (Eugine, 2015)*

Countries	Data source
Canada	Furlong (2007) and McCormack (2005)
South Africa	(Lambert, 2002)
New Zealand	(McKenzie and Wegelin, 2009)
Australia	(Lambert and Taylor 2010)
Austria	(Koelbl et al., 2009)
Asia	(Asia Development Bank, 2011)
Africa	(WSP, 2009)
Latin America	(van Den Berg and Danilenko, 2011)

Using Data Envelopment Analysis (DEA) to benchmark Indian water utilities, Singh et al. (2010) found inefficiencies in water distribution systems that could be improved to reduce NRW further. Balkaran and Wyke (2002) found that inefficient water distribution systems in Palestine were caused by water losses. A similar benchmarking initiative was done in America on 100 water utilities using linear regression. The findings showed that systems efficiencies were correlated to

WLM strategies adopted (Park, 2006). Benchmarking initiatives rely on legitimate data; otherwise, other methods may be utilized to identify causes of inefficiency in water delivery systems.

2.10.5 District metered area management Approach

When establishing a district metered area (DMA), it's important to assess NRW values, Net Night Flow (NNF), and commercial losses to identify areas of concern (USAID and WBI 2010). If DMA leakage is high, execute appropriate NRW reduction operations. Farley et al. (2008) found that the NRW level in a DMA varies with infrastructure variables such as pipeline age, network wear and tear, and system pressure dynamics. The water utility is responsible for monitoring the key components of NRW, including physical and commercial losses (Eugene, 2015). The formula for NRW within a DMA is as follows:

$$\text{DMA NRW} = \text{Total DMA Inflow} - \text{Total DMA Consumption}$$

A DMA's level of metering has a general effect on water consumption. Higher water meter density results in higher usage. If all users in a DMA are metered, the total consumption is the sum of all individual meters. For partially metered systems, DMA consumption is approximated by per capita consumption. Additional information on water demand and average per capita values is required.

3 MATERIAL AND METHODOLOGY

3.1 The Study Area

Addis Ababa was founded in 1889 and is located (38.73° E and 9.17° N). According to World Population Review, Addis Abeba is home to more than 20% (5,460,591) of Ethiopia's urban population. The city's covers total area 520 km², as shown in Figure 3-1 with elevations ranging from 2000 to 3000 m above mean sea level (amsl). The average annual maximum temperature in the city is around 24 °C, and the average annual minimum temperature is around 12 °C. The average monthly rainfall in July and August is substantial (about 260 mm). The average annual rainfall in Addis Ababa is around 1255 mm (B. Kifle Arsiso *et al.*).

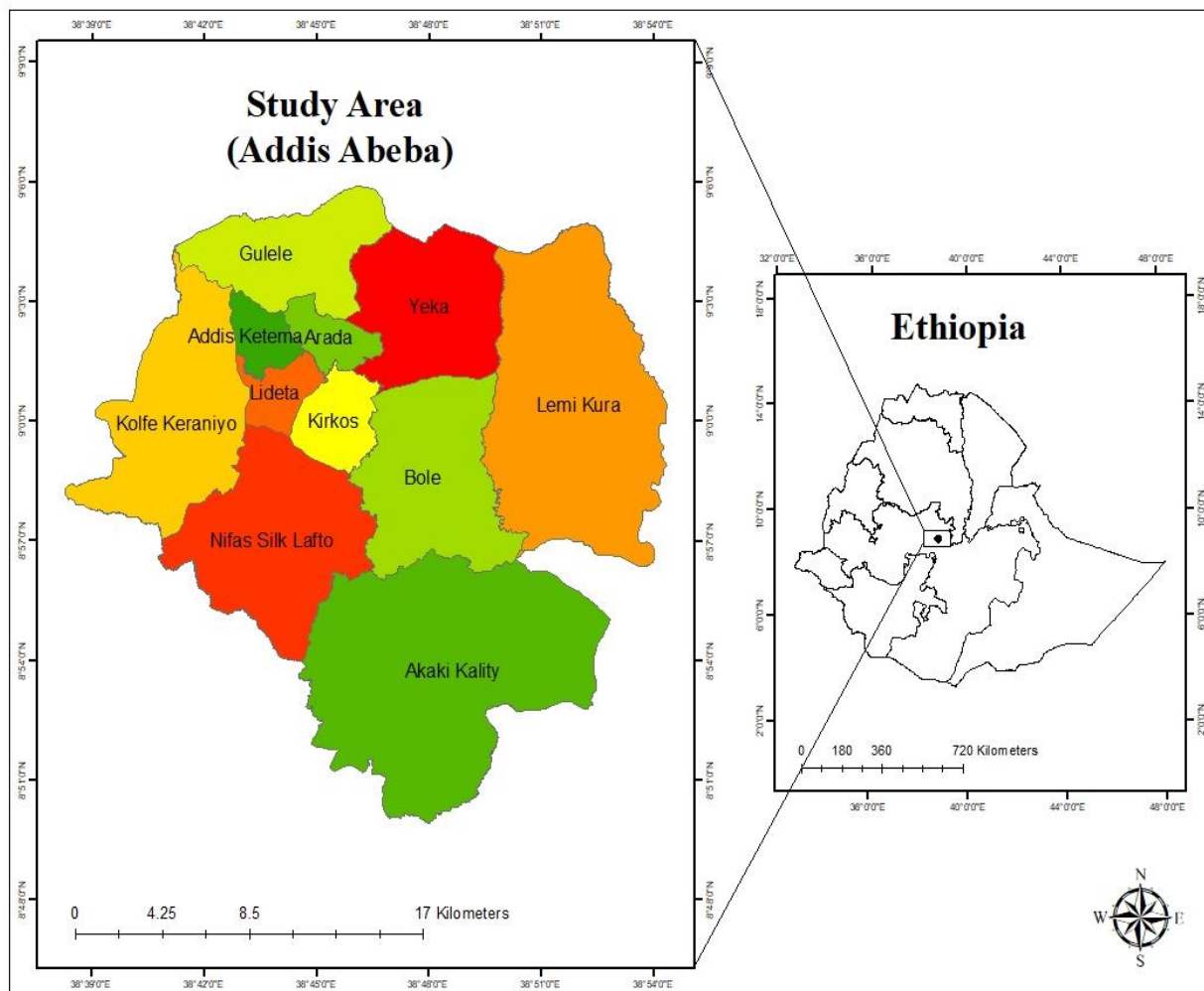


Figure 6 Map of the study area

3.2 Addis Abeba Water Distribution System

Addis Abeba is the capital city of Ethiopia and is the biggest branch utility managed by AAWSA. The Addis Ababa Water and Sewage Authority (AAWSA) was founded in 1971, and the only water service provider for the city with two missions in consideration: to provide potable water to the city and to operate a sewerage system for the safe disposal of sewage. According to AAWSA, over 5 million people live within the service region. Addis Abeba is one of the world's fastest-growing cities, with an annual population growth rate of 3.8% since the 1991 census. Like in most developing countries, water supply infrastructure development has not kept pace with population growth and has resulted in water shortages and low pressures in most parts of the distribution system. To close the gap, AAWSA has invested heavily in augmentation projects (increased water production) rather than water loss reduction programs.

The provision of piped-water supply was started during the reign of Menelik II to serve a few residents of Addis Ababa. Initially, the numerous springs and wells located at the foot of the Entoto Mountain were used as the source of this water supply. The eventual growth of the city's population and subsequent increase in the water demand necessitated the creation of a Water Department under the city's Municipality Office. The main source of water was also shifted to the Gefersa Dam, which was constructed in 1942/43 (AAWSA, 2011) (Supply et al., n.d.).

Subsequently, Gefersa II and III reservoirs - built in 1960 and 1966 - increased water supply. While the Gefersa II development was an expansion of the Gefersa I reservoir, the Gefersa III reservoir was a separate dam constructed about 800 m upstream of the Gefersa I and II reservoirs. The Gefersa reservoir system has an area of 130.5 ha and a catchment area of 53.5 km². The Gefersa reservoir system has a water storage capacity of ~8 Mm³ and supplies 30,000 m³ /day of water to Addis Ababa (AAWSA, 2011).

Legedadi reservoir, commissioned in the 1970s, has a capacity of 50,000 m³ /day of water (Supply et al., n.d.) and a 510-ha surface area with a 207.3 km² catchment area. The Dire reservoir - built in 1998 - expanded the Legedadi water supply system to meet Addis Ababa's increasing water demands. The Dire reservoir has a 165-ha surface area and a catchment area of 77.5 km². The combined Legedadi and Dire water supply system has a water storage volume of 120 Mm³ (86 and 34 Mm³, respectively) and supplies 165,000 m³ /day to Addis Ababa City (AAWSA, 2011).

The Akaki well fields, springs, and deep wells also serve as groundwater sources for the city. These groundwater sources supplied more than 75,000 m³ /day of water by 2010 (AAWSA, 2011) and significant investments have occurred in groundwater borehole development. For example, the new Akaki well field, which supplies 73,000 m³ /day of water, was commissioned in 2012 (AAWSA, 2012). Further developments in spring sources and deep wells have occurred in various parts of the city since 2012. The AAWSA report of 2023 showed that the city is supplied water from Legedadi and Gefarsa treatment plants and from boreholes drilled in the different parts around and within the city areas and also from Pocket Boreholes within the city. This makes a daily total production of 575,000 m³/d. However, the estimated current demand of 1,064,814 m³/day. This situation is exacerbated by the high water losses in the Water Distribution System with NRW estimated at 43% of SIV (or 247,250 m³/d) (AAWSA, 2023).

Figure 2.2 Shows the NRW trends over the last eleven years. In the Addis Abeba City distribution network, the highest NRW loss was observed at 46.9% in 2009. For the years 2003 (34.43%) to 2005 (44.87%), there was a constant increase with a rate of (about 5%) in NRW. There was variation in NRW over the years. This NRW fluctuation could be attributed to a lack of planning and methods to reduce NRW losses more sustainably. Water loss reduction requires a coordinated approach that addresses technical, operational, institutional, financial, and administrative issues (H. E. Mutikanga et al., 2009).

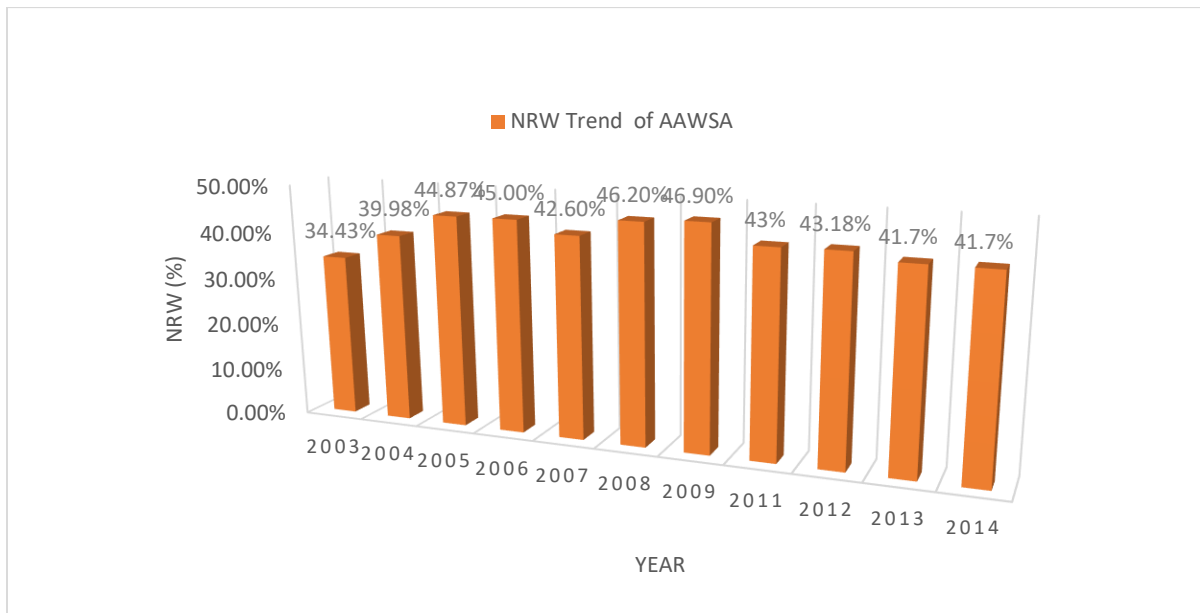


Figure 7 NRW trends for AAWSA

The AAWSA encompasses a service area of about 520 km² covering the city. It serves about 615,372 customers (83% domestic and 14% commercial) through 3851 Km of pipelines (Supply et al., n.d.). The old pipelines (over 50 years) are mainly made of steel and cast iron while the newer parts of the mains consist of plastic pipe materials. water transmission and distribution pipe diameters range from 50 mm to 1400 mm. There are 99 reservoirs located in various parts of the system with total storage capacity of about 183,018 m³. The reservoirs are used not only as pressure stabilizing tanks to supply the night flow as well as peak hour demands, but they are also being used as boosting reservoirs to elevate the water to relatively higher elevations and in certain cases as pressure break tanks. The distribution system of Addis Abeba has 35 booster stations with 127 surface pumps that are used to elevate the reservoir water to the next highly elevated ones (such as Terminal, Ras Hailu, Teferi mekonnen) and at some stations (such as at Balcha Hospital and Urael pumping stations) the water in the transmission system is boosted to induce additional pressure in the pipes. In some places, direct pumping into the distribution system is also being exercised.

3.2.1 Existing and newly proposed Addis Abeba Water utility area

Currently, there are 8 existing utility branch boundaries and one newly proposed utility area that sub-divided the city of Addis Ababa for the efficient and effective operation and service provision

of water supply and wastewater collection and disposal by AAWSA. The boundaries are delineated based on the natural boundary. From the existing GIS Map and list of services, one branch office serves more than one administrative sub-city except for the Akakai Kality branch office, which is limited within the sub-city boundary area. Table 11 shows the sub-cities being served by each existing utility branch boundary.

Table 11 Sub-cities covered or served by each existing branch utility boundary

No.	Branch Name	No. and Sub-city Name Served by the existing Branch
1	Addis Ketema	4 Addis Ketema, Gulele, Kolfe Keraniyo, & Lideta
2	Arada	5 Addis Ketema, Arada, Kirkos, Lideta & Yeka
3	Akaki Kality	1 Akaki Kality
4	Gurd Shola	3 Bole, Lemikura, & Yeka
5	Gulele	3 Arada, Gulele & Yeka
6	Lemikura	1 Lemikura
7	Megenagna	3 Bole, Kirkos & Yeka
8	Mekanisa	4 Kirkos, Kolfe Keraniyo, Lideta & Nifas Silk Lafto
9	Nifas Silk Lafto	2 Kirkos , & Nifas Silk Lafto

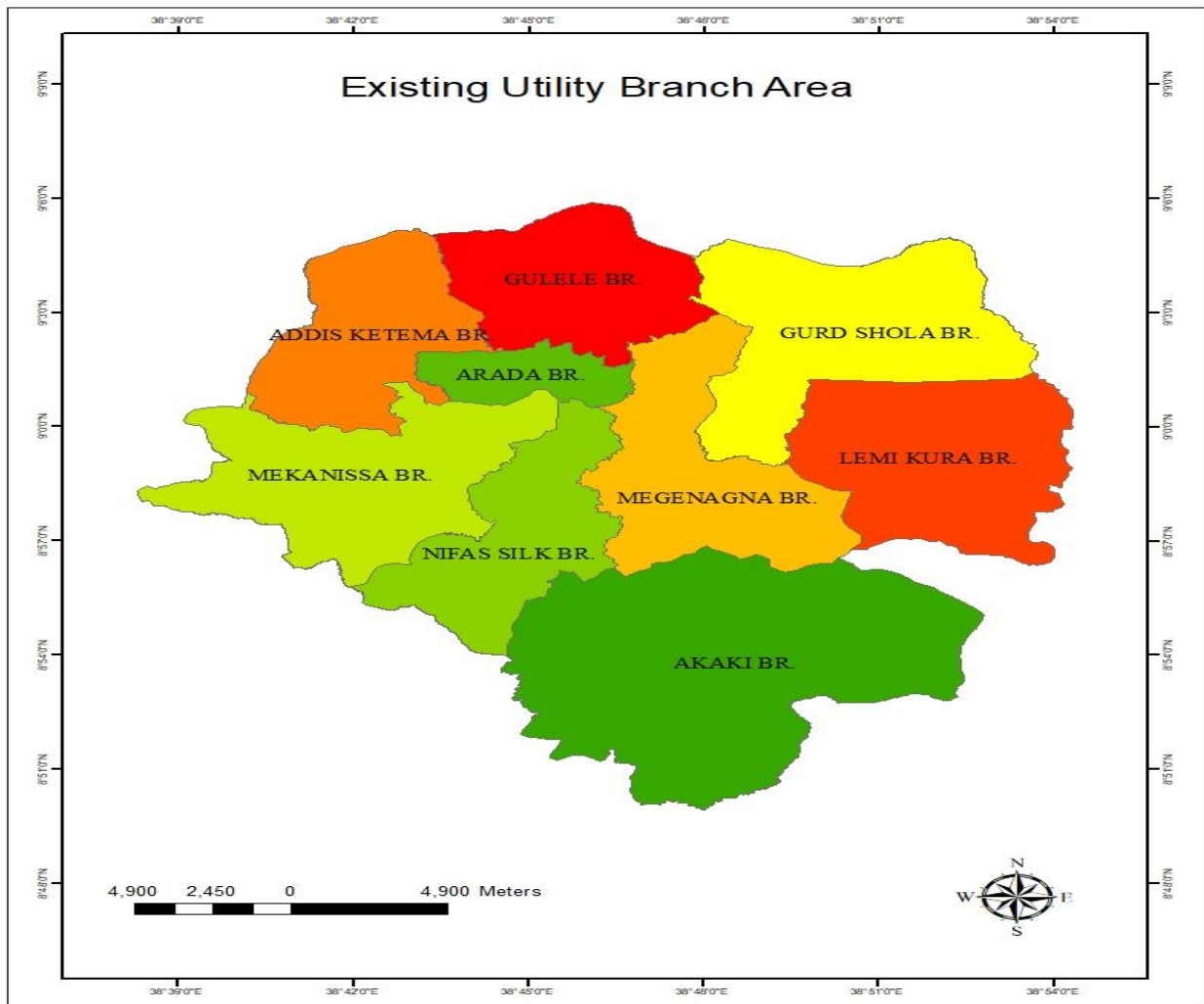


Figure 8 Existing Branch Utility Area

3.3 Mode of Water Supply

There are three water supply modes in Addis Abeba. The first is the pumping mode, where the areas are supplied off primary pumping mains. Second, there is the gravity supply mode where areas are supplied from storage tanks by gravity. The last category is supplied from both pumping and gravity. Various pressure zones were created in Addis Abeba but owing to system failures, most zone valves have been opened and this is exacerbating water losses.

3.4 Study Design

The study used literature review and field work in gathering important information. The data collection was carried out on Addis Abeba water distribution system from March, 2022 to April, 2023. Selection of Addis Abeba to be the study area was based on the reporting that it has high levels of NRW of up to 46.9% of water production. On the other hand, the contribution made by water leakages from distribution mains was unknown. The components studied include efficiency measurement of water distribution system using IWA performance indicators; investigate customer water meter performance mainly focus on small meters of size 15 mm; the influence of different factors affecting real water losses and apparent water losses; water leakage contribution to the water losses in selected DMAs. DMAs were selected based on their unique characteristics in the sense that they were distinct metered zones which do not supply water to any other areas.

3.5 Materials

Table 12 The Materials Used in this study

	Tools	Purpose	Source
1	Water GEMS V10 Software	For hydraulic analysis in the distribution system	Downloaded from https://bentleywatergems.software.com
2	ArcGIS 10.8 Software	For mapping the study area, a Water Customer Point Shape file was created, and to visualize the water distribution layout	Downloaded from www.ESRI.com
3	AutoCAD 2007 Software	To extract AutoCAD drawing elements that combines graphics and data.	Downloaded from https://Softvela.com
4	Metrolog pressure logger of type MET-P V2/P/F/D/20	To measure Pressure Data for DMA	From Addis Abeba Water Supply and Sewerage Authority Arada Branch

3.6 Data collection

The data was involved both primary and secondary sources. For this study, the primary data were obtained from pressure reading, and flow reading. Field measurements (flows and pressures) were carried out. Pressured was measured during a 24-hour period in 3 locations in the network using the Metrolog pressure logger of type MET-P V2/P/F/D/20. This data used for the calibration and validation purposes to check the robustness and precision of the model.



Figure 9 Data logger for pressure measurement

While, secondary data were collected from different literature reviews, design report, the city water supply service office existing documents, from branch office and annual reported papers. Input data for network assets, boundary of the DMA, computer-aided design (CAD) drawings obtained from Arada Branch utility office, and Digital Elevation Model (DEM) data obtained from Space Science and Geospatial Institute. The total population to be served and the total number of houses, as well as the number of customer data per level of connection data was taken from AAWSA. Have also obtained successive water production and consumption data from AAWSA.

3.7 Asses and quantify the water loss in Water Distribution System

3.7.1 Assessment of Apparent losses in Water Distribution System

Apparent losses (AL) are the non-physical component of water losses in the distribution system. They include all types of inaccuracies associated with customer metering, data handling errors (meter reading and billing) and unauthorized consumption (theft or illegal use). The customer meter inaccuracies are often thought to be the main cause of apparent losses. Like all mechanical devices, mechanical water meters typically decline in accuracy with usage over time, causing substantial revenue losses to the utility (H. E. Mutikanga et al., 2013). To minimize these losses, many researchers have developed tools and methodologies for water meter replacement based on meter testing, economic optimization, and operational research techniques ((H. E. Mutikanga, Sharma, et al., 2011b); (Yee, 1999); (Noss et al., 1987); (H. E. Mutikanga, 2012); (F. J. Arregui et al., 2011)). Although the tools and methodologies developed are valuable, their application in practice is rather difficult for various reasons in case of Addis Abeba city such as (1) It is difficult to remove in-service water meters for laboratory testing. There are about 615,372 installed customer meters. Testing all of these meters and measuring consumption patterns can only be realized by defining and establishing representative samples of at least 5% (30,768) was not possible (2) labor intensive (3) expensive to implement (4) the process is too costly. In this study applied Meter accuracy degradation rate and Meter Failure.

AAWSA has number of service connections with meters of size 15 mm is about 82.2% of the total service connections (or 505,872 service connections). These connections thus represent a sizeable investment for the utility and are mainly domestic users and small commercial customers. There are 2 types of in-service meter models for this size of meters: velocity type or inferential meters (multi-jet and single-jet) and volumetric or displacement (oscillating-piston type) meters. The average age of the meters is about 10 years, with some meters being more than 20 years old. Until recently there was no proactive meter sampling, testing, and replacement. Only reported defective and vandalized customer meters were replaced. However, meter management has greatly improved in recent years, and critical meter testing equipment at the meter laboratory has been upgraded.

3.7.1.1 Quantification for Apparent Water Loss due to Meter accuracy degradation rate and Meter Failure

To manage water meters better, it is important to quantify water loss due to Meter accuracy degradation rate and Meter Failure. Until now, the procedures for estimating these losses are not yet well established. In this section, estimating these losses is presented.

3.7.1.1.1 Quantifying Apparent water losses due to metering Failure

The procedure for estimating water loss is adapted from (H. E. Mutikanga, 2012) and is shown below.

1. Estimate meter failure rate (MFR) by dividing the total number of meter failures per year by the total number of in-service meters.
2. Calculate the average annual water used (Q) through the meter from the customer billing database.
3. Establish the total number of metered active connections (MAC) from the customer billing database. If all service connections are metered and on supply, then MAC is equal to the total number of service connections (TSC).
4. Estimate the average detection-replacement/service time (DRT) for failed meters based on utility meter reading cycles (MRC). If MRC is say one month, meter failures would be expected to exist for 15 days on average and 2 weeks would be a reasonable time within which to service or replace the meter. DRT in this case would be about 1-month.
5. Estimate total unregistered water due to meter failure (UWF m³/yr) from Eq. 3.1.

$$UWF = MFR \times MAC \times Q \times (DRT/12) \dots\dots\dots (3.1)$$

3.7.1.1.2 Quantifying Apparent Water Loss due to Meter accuracy degradation rate

The comparative billing analysis methodology proposed by (F. Arregui et al., 2003) & (Mbabazi et al., 2015) which relies on existing data and not on costly fieldwork and statistical studies was the basis of the research. Statistical sampling tools (stratified random sampling) and regression analysis techniques were applied to group meters in order to determine sample sizes and to develop the meter accuracy degradation profiles.

(Mbabazi et al., 2015) estimated meter accuracy degradation rates by evaluating the evolution of the average yearly metered volume versus the age of volumetric and velocity meters in Uganda.

The obtained rates were used to estimate the losses based on the consumption patterns of the utility. The analysis evaluated three 15 mm meter models with meters grouped according to model types and ranges of the total accumulated volume on each meter. A total of 503 meters were eventually used from a dataset of 128,201 meters, with 8 months of consumption data. The average metered volume for each grouping of the accumulated volume for the meters was calculated. Using the age of the meter, evolution curves of the average yearly metered volume (annual billed volume) versus age for each model were thereafter derived. The relationships for the average yearly metered volume and meter age and the degradation rate are given (Mbabazi et al., 2015) :

$$Y = \beta_0 + \beta_1 x \dots \dots \dots (3.2)$$

Where y is the annual billed volume and x is the age of the meter and the degradation rate d is calculated as:

$$d = \frac{\beta_1}{\beta_0} \dots \dots \dots (3.3)$$

In this study, only meters of DN of 15 mm were considered. This is because they constitute about 82.2% of the meters installed in Addis Abeba water distribution system according to AAWSA, 2023. Three different meter models were examined: (1) two types of volumetric meter models (Model 1 and Model 2) from two different manufacturers; and (2) a multi-jet meter type (Model 3). Models 1 and 2 constitute in the network and make up 49.4 % of all the small meters of size 15 mm; 18.3 % are of the velocity type of size 15 mm.

There are several types of volumetric water meters available, including turbine meters, piston meters, and nutating disk meters. Each type has its own advantages and is suitable for different applications based on factors such as flow rate, pipe size, and water quality.



Figure 9 Volumetric Rotary piston water meter *Source: (Jiangbei manufacturing)*

Volumetric water meters operate based on the principle of displacement. Water flowing through the meter causes a moving mechanism, such as a turbine or piston, to rotate or move. The rotation or movement is directly proportional to the volume of water passing through the meter.

In sampling of the meters, meter data (model and DN), monthly consumption and meter cumulative volume data were taken from the utility's billing database, and meters were grouped based on model and total accumulated volume. Strata of meters were based on meter models. Within each stratum, the meters were grouped based on cumulative volume to build more homogenous groups and reduce variability associated with sampling. Sample size was selected according to The Research Advisors (2006) sample size table recommendations at 95% confidence level considering a 5% margin of error for each sub-stratum in order to have a more reliable statistical judgment derived from sample collection. The age of the meters was determined based on monthly water consumption (water meter reading taking regularly every month) because there was no data when meters were first installed in the field. The age of the meters was calculated from two variables, i.e., total cumulative volume (m^3) and current consumption (m^3) as shown below:

$$\text{Age} = \left(\frac{\text{total cumulative volume } (m^3)}{\text{current consumption } (m^3)} \right)$$

Analysis of the billing database

Data for meters of size 15 mm were obtained from Addis Abeba Water and Sewerage Authority (AAWSA) billing database for 3 years to analyze the meter accuracy degradation rate. The data obtained consisted of 21-meter with different manufacturer. However, the most dominant meter models, constituting were Models 1, 2, and 3. Of these, 133,680 were Model 1 meters, 116,069 were Model 2, and 92,414 were Model 3. Meters were grouped into five sub-strata (0–2,000, 2,000–4,000, 4,000–6,000, 6,000– 8,000, and more than 8,000 m³) based on cumulative volume through the meter. The last group was set to more than 8,000 m³ because the expected meter life for a half inch meter is when the odometer reading clocks 8,000 m³ (H. E. Mutikanga, 2012). Upon stratification and screening only 825 meters were selected for the study. the sample sizes selected were 447 for Model 1, 269 for Model 2, and 122 for Model 3 meters. Screening of the data involved eliminating no registered volume (zero current reading) assumed to be newly installed meters and nonfunctional (stuck) or abandoned meters were excluded from the analysis. Additionally, water meters with meter abnormal spikes or dips in consumption (abnormally high or low volumes) caused by poor data handling from capturing data to the customer billing data base were also excluded.

For each of the meter reading range categories, the average annual consumption per age category was calculated to determine the variation of average annual consumption with age. The analysis was done in MS Excel to obtain regression curves for each of the ranges, including key statistics that could be used to estimate potential apparent water losses. The water volume registered by a meter each year (hereafter, annual billed volume) was calculated from two variables, i.e., last reading of the totalized volume (m³) and age given by (Musaazi et al., 2021)

$$\text{Annual billed volume (m}^3\text{/year)} = \frac{\text{last reading of totalized volume (m}^3\text{)}}{\text{Age (year)}}$$

Determination of meter accuracy degradation rates

Graphs of the annual billed volume against total age of the meter were analyzed to determine the meter accuracy degradation rates.

Water meter accuracy degradation is a function of many variables (pressure, water quality, leaks, and users' storage tanks and age). In these studies, different researchers (H. E. Mutikanga, 2012);

(F. . Arregui et al., 2006); and (Mbabazi et al., 2015) assumed a linear relationship between accuracy and age of the meter for domestic meters. In this research, a regression analysis with a linear relationship to predict meter accuracy degradation rate was assumed due to its simplicity and also because it is cheaper, involves less time and technical requirements and thus can be afforded by small utilities. The regression model was performed for specific meter models (same manufacturer, meter size, and metering technology).

Testing of the meter accuracy degradation rates using the optimal consumption patterns to determine annual water losses is given by (Mbabazi et al., 2015):

$$W = DP \dots\dots\dots (3.4)$$

where D was the meter accuracy degradation rate, W was the annual unmeasured water, and P is the average annual water consumption which was obtained from the billing database

3.7.1.2 Assessment of Systematic Data Handling Errors

Systematic Data Handling Errors, according to AWWA, occur as a result of customer consumption volume and can result in a direct loss revenue potential. It includes all forms of errors, such as meter reading error, billing error, archiving, and reporting.

3.7.1.2.1 Assessment of meter reading errors

AAWSA uses the traditional method of meter reading, in which meter readers visit individual meters and manually collect monthly readings. This method of reading meters is prone to human error, especially when readings are taken in a hurry to achieve meter reading targets.

Meter reading errors was carried out for Addis Abeba city’s distribution system using the methodology used by (H. E. Mutikanga, Sharma, et al., 2011a) . During the month of March 2023, meter reading audits based on random sampling were performed (in different parts of the network) in a day to verify the accuracy of meter readings given by meter readers for billing purposes. The collected meter readings were then compared to the values given by meter readers. Readings with unrealistic variations were considered erroneous. The billed consumption volume based on erroneous meter readings was then summed (z m³) and reported as a percentage of volume of water sold (y m³) for the total audited accounts. The outcome is the meter reading error (z/y X

100), which is meant to be representative of the entire system. The total number of meters audited was 6154 (about 1% of all customer meters).

3.7.1.2.2 Assessment of Data Handling and billing errors

These errors occur during the transfer or capture of data from meter reading sheets into the customer billing database. Gaining access to some customer meters installed inside customer premises is challenging due to an increase in the number of working couples who leave no one at home or, in some cases, leave guard dogs. Furthermore, there is an increasing number of defective customer meters, as well as a lag in updating new connections in GIS, which complicates tracing meters placed. As a result of these factors, manual meter reading success rates in Addis Abeba are declining. Customer water consumption is then estimated using historical consumption trends. While this is a reasonable approach, multiple cycles of meter reading without an actual reading greatly increase the prospect of inaccurate estimates (AWWA 2006; Thornton et al. 2008).

A data-capturing audit was carried out to compare the input data used for billing and the readings on the meter reading sheets submitted by meter readers. The readings that were wrongly captured in the billing database were established and their corresponding total volume was computed ($x \text{ m}^3$). If the water sales for the assessment period were $y \text{ m}^3$, the percentage data handling errors were computed as $(x/y \times 100)$. A sample of 4774 customer accounts was analysed.

Billing errors from poorly estimated volumes that resulted in billing adjustments were generated from the customer billing database and their volume was summed up ($v \text{ m}^3$). If the water sales for the assessment period were $y \text{ m}^3$, the percentage data handling errors were computed as $(v/y \times 100)$. Billing errors for the 6 months (September 2022 – March 2023) were used for computation of billing errors' component of apparent losses.

3.7.2 Assessing and quantifying leakage in the water distribution system

Measuring losses within the distribution network is more difficult since it usually has a more complex structure. Measurements can be best performed if supply arrangements can be temporarily changed and the supply area can be separated into discrete zones with designated boundaries (Fallis et al., 2011). In this Study, Figure 10 shows the selected district metered area (DMA). The DMA is located on bordering Kebena River. The selected DMA was assumed to be representative of the entire network characteristics of AAWDS with respect to water supply

Regime (intermittent supply), leakage distribution and infrastructure age when compare to the other DMAs shown in Table 13.

Table 13 DMAs Characteristics

	DMA1 found around megenagna	DMA 2 found around bulbula	DMA 3 found around kasanchis	DMA 4 found around 72 area
Set of criteria should satisfy (Farley and Trow 2003)				
Natural geographic boundaries and less artificial boundaries	Satisfied	Satisfied	Satisfied	Satisfied
Hydraulically separable or isolated	Satisfied	Satisfied	Satisfied	Satisfied
Leaks and bursts were reportedly common	Not Satisfied	Not Satisfied	Satisfied	Not satisfied
Single inlet and outlet	Not Satisfied	Not Satisfied	Satisfied	Satisfied
AAWDS network Characteristics				
Water supply regime (Intermittent Supply)	Not Satisfied	Satisfied	Satisfied	Not Satisfied
Mixed infrastructure age	Not satisfied	Not Satisfied	Satisfied	Not Satisfied

DMA was created from a larger network zone. The selected DMA satisfied the following set of criteria (Farley and Trow 2003):

- Ø Natural geographic boundaries and less artificial boundaries;
- Ø Hydraulically separable or isolated;
- Ø Leaks and bursts were reportedly common;

Ø single inlet and outlet

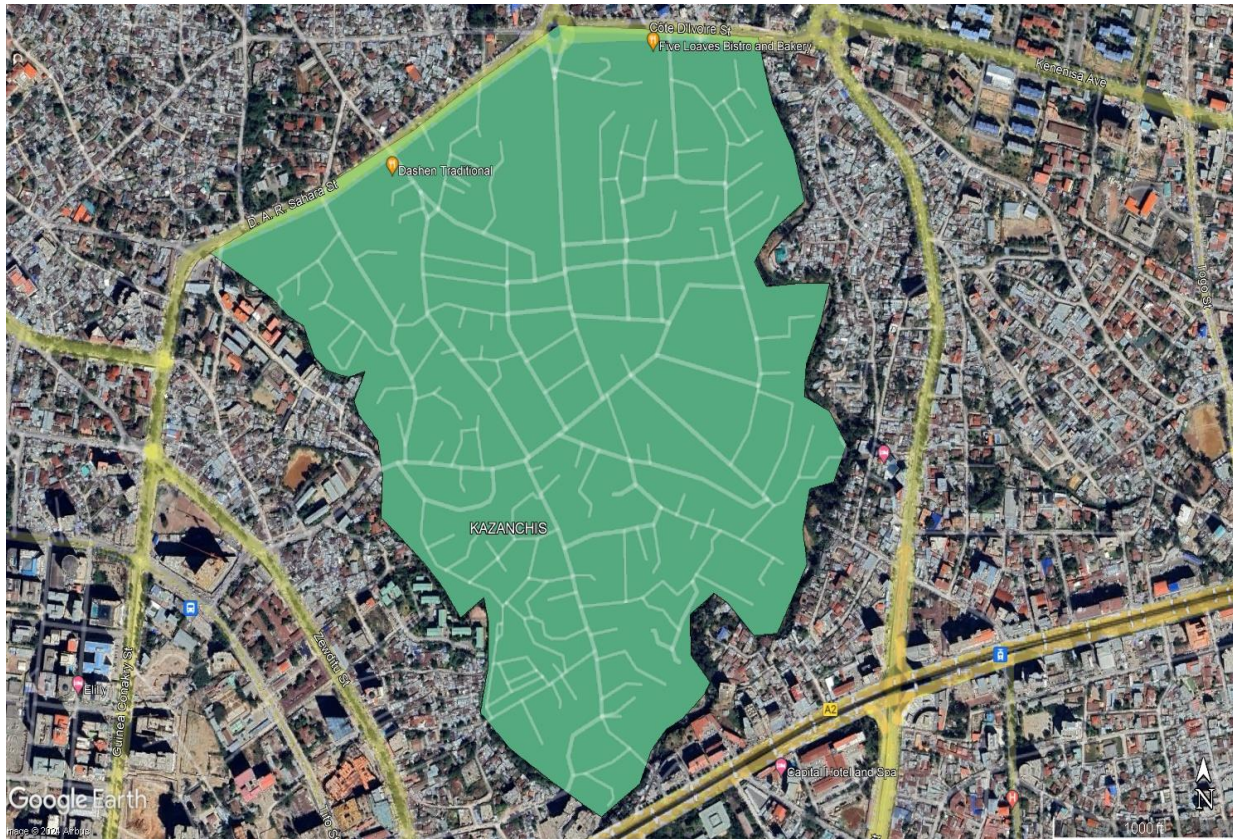


Figure 10 The Selected DMA

In this section, different methodologies are applied to assess and quantify leakage in water distribution system for a selected DMA. The model (Figure 10) shows a district metering area (DMA) represents a real network. The DMA has a total of 2415 Customers, 2110 Domestic customers, and 305 Non – Domestic customers. Total pipe length is 14.389 kilometers, with diameters varying between 40 to 300 mm.

In DMA, Leakage estimation was based on two methodologies: the UK “bottom-up” MNF analysis (Farley and Trow 2003) and (Network Hydraulic Modeling) advanced computational methodology for estimating leakages from calibration process.

3.7.2.1 Quantifying leakage based on Minimum night flow method

This method assumes that leakages in a District Metering Area determined when the flow into the DMA is at its lowest. It usually happens at night between 2 am and 4 am., when consumer demand is at its lowest and the leakage component accounts for the greatest proportion of the flow (Environmental Protection Authority, 2007) & (Cheung et al., 2010).

In order to accurately measure night flows and apply MNF concepts for leakage assessment, special arrangement was made to ensure 24-hour supply to DMA1 during the study period. Although MNF analysis is not suitable for systems with intermittent supply, usually, most storage tanks are full between 3.00 am and 5.00 am, the period when MNF field tests are carried out and most users are hardly using any water. For the same reason, MNF analysis has been used in other developing countries with similar supply regimes e.g. Brazil (Cheung et al., 2010). In order to determine the DMA leakage, night flow measurements were carried out and MNF assessed. In DMA1, the highest MNF was 183 m³/h and the lowest MNF was 91 m³/h, while the average MNF was 130 m³/h. The common feature surrounding these flows is that, they all largely occurred during the same hour from 2:00 to 4:00 am, when system pressure was at its maximum. Therefore, the hour at which pressure dependent leakage was at its maximum during the day, was also from 2:00 to 4:00 am. The lowest measured value of MNF, which is an indicator of leakage, was the value closest to the actual night leakage rate. Hence, MNF in DMA1 was taken to be 91 m³/h.

To calculate leakage at MNF time (Q_L), Equation 3.5 was applied to the DMA (Farley and Trow 2003).

$$Q_L (t_{MNF}) = Q_{DMA} (t_{MNF}) - \text{Legitimate Night-Time Uses} \dots \dots \dots (3.5)$$

For estimation of legitimate night uses, detailed field investigations are required. In the absence of such detailed studies, McKenzie (2001) proposed use of 6% of total population and average use of 10 L/person/hour at time of MNF. However, these are default values for South African conditions and may not be valid for other countries. The default value of 10 L/person/hour depends on toilet flush capacity. (H. E. Mutikanga, Vairavamoorthy, et al., 2011) in study used 10 L/property/hr for (non-households) and 3 L/household/hr for Kampala, Uganda condition. The water use is low when compare to McKenzie (2001) proposed value this is because (H. E. Mutikanga, Vairavamoorthy, et al., 2011) in the study considered 10% as the active population coefficient,

and most households are in urban poor settlements where houses lack internal plumbing. Use of pit latrines instead of flush toilets is the norm for most households. The use of active population percentage at time of MNF depends on socio-economic life styles of the population and probably on level of urbanization. Assessment of legitimate night-time use is crucial for accurate leakage reduction predictions. Over-estimation will lead to low leakage levels while under-estimation will lead to high leakage levels, thus exaggerating potential water savings.

In this study, estimation of the average night consumption for a households and non-households was not possible due to inadequate data logging equipment and the high number of properties involved beside that the ideal way is to physically read the individual meters for the non-households during the hour of MNF as their consumption do vary a lot which was difficult.

The daily level of real losses obtained from the minimum night-time flow analysis can be determined by applying the FAVAD principles and simulating leakage over the full 24h period (Environmental Protection Authority, 2007), (Cheung et al., 2010) & (H. E. Mutikanga, Vairavamoorthy, et al., 2011)

Daily real loss volume (DRLV) determined by using Equation below (Cheung et al., 2010)

$$DRLV = F_{nd} \times Q_{mn} \dots \dots \dots (3.6)$$

Where Q_{mn} is the average minimum nightly leak flow rate (m³/h) and F_{nd} is called night-day factor. It supposes that the leakage volume is not constant during day because depends of demand pattern. The F_{nd} is computed by the sum of pressure values acquired during 24 hours in an average DMA representative point, by using the following relation:

$$F_{nd} = \sum_{i=0}^{24} \left(\frac{P_i}{P_{3-4}} \right)^{N1} \dots \dots \dots (3.7)$$

where P is the average pressure in one observed point of a DMA for each i time; P_{3-4} is the average pressure during minimum nightly consumption between 3 : 00 and 4 : 00 am; N1 is the orifice exponent that can be computed using the following relation:

$$\frac{Q_1}{Q_0} = \left(\frac{P_1}{P_0} \right)^{N1} \dots \dots \dots (3.8)$$

where Q_0 is the flow rate in association with P_0 pressure; Q_1 is the flow rate in association with P_1 pressure and $N1$ is obtained by closing the valve usually situated in system entering. Experiments performed in several countries have determine $N1$ close to 0.5 for metallic pipes $N1$ value for non-metallic pipe close to 1.5 or more. (Environmental Protection Authority, 2007)

3.7.2.2 Quantifying leakage using Water GEMs

3.7.2.2.1 Network Hydraulic Modeling (NHM)

The network hydraulic model of the pilot study area (DMA) was built using input data for network assets obtained from the utility. Pipe data: Pipe diameter, Pipe length, Pipe Material. computer-aided design (CAD) drawings, and Digital Elevation Model (DEM) data obtained from Space Science and Geospatial Institute with twenty-meter interval in order to generate elevation data for the nodes. The NHM was built using Water GEMs v10 hydraulic software. The case study network model (DMA) is shown in Figure 3.3. The network model for DMA contained a reservoir, 127 pipes and 118 junctions. The model was applied for Extended Period Simulation (EPS) under steady-state conditions. To enable EPS, hourly demand multipliers or peak factors, derived from the diurnal flow profile at the DMA inlet, were used for the hydraulic analysis. A 24-hour diurnal cycle was applied for modeling.



Figure 11 Network Model layout of DMA

3.7.2.2.2 Modeling leakage using the emitter coefficient

Total flow into the DMA was split into two components: nodal outflows (pressure-dependent leakage) and nodal demands (pressure-independent). This split is rather subjective and based on simplified assumptions. In Water GEMS, emitters are used to simulate leakage in a pipe connected to the junction. In order to perform hydraulic simulations, leakage was incorporated in the models using the emitter devices. Flow Emitters are devices associated with nodes that model the flow through a nozzle or orifice. In these situations, the demand (i.e., the flow rate through the emitter) varies in proportion to the pressure at the node raised to some power as outlined by various researchers (Gajbhiye et al., 2017) & (H. E. Mutikanga, Vairavamoorthy, et al., 2011). The emitter nodes allow leakage to be modeled using appropriate pressure-dependent outflow relationships as shown in equation

$$Q = k \times P^n \dots\dots\dots(3.9)$$

Where, Q is the leakage flow; k is emitter coefficient; P is pressure; n is emitter exponent (default 0.5 for orifice).

3.7.2.2.3 Thiessen polygon method of calculating nodal demand

The allocation of base demand is critical for accurate modeling. The base demands were obtained from the utility customer billing database allocated by aggregating consumptions to the nearest nodes. This was enabled by created node service polygon. Junction nodes are point features, but some demand allocation methods require that the nodes have a polygon service area associated with them. Thiessen's polygons were generated for each node in the Water gems V10. Thiessen polygons define the individual tributary areas for each node. The space is divided such that any point within a particular Thiessen polygon is nearer to that polygon's node than to any other node. The point shape file (house service connection demand) a total of 2415 water customer were created in ArcGIS 10.2 for the DMA, all house service connection demand which are falling in one polygon, the total demand was added and assign to the node. This was enabled by integrating GIS block map information with unique customer account references that are also used in the customer billing database. It was assumed that nodal consumption followed the same pattern as the diurnal flow profile measured at DMA inlet.

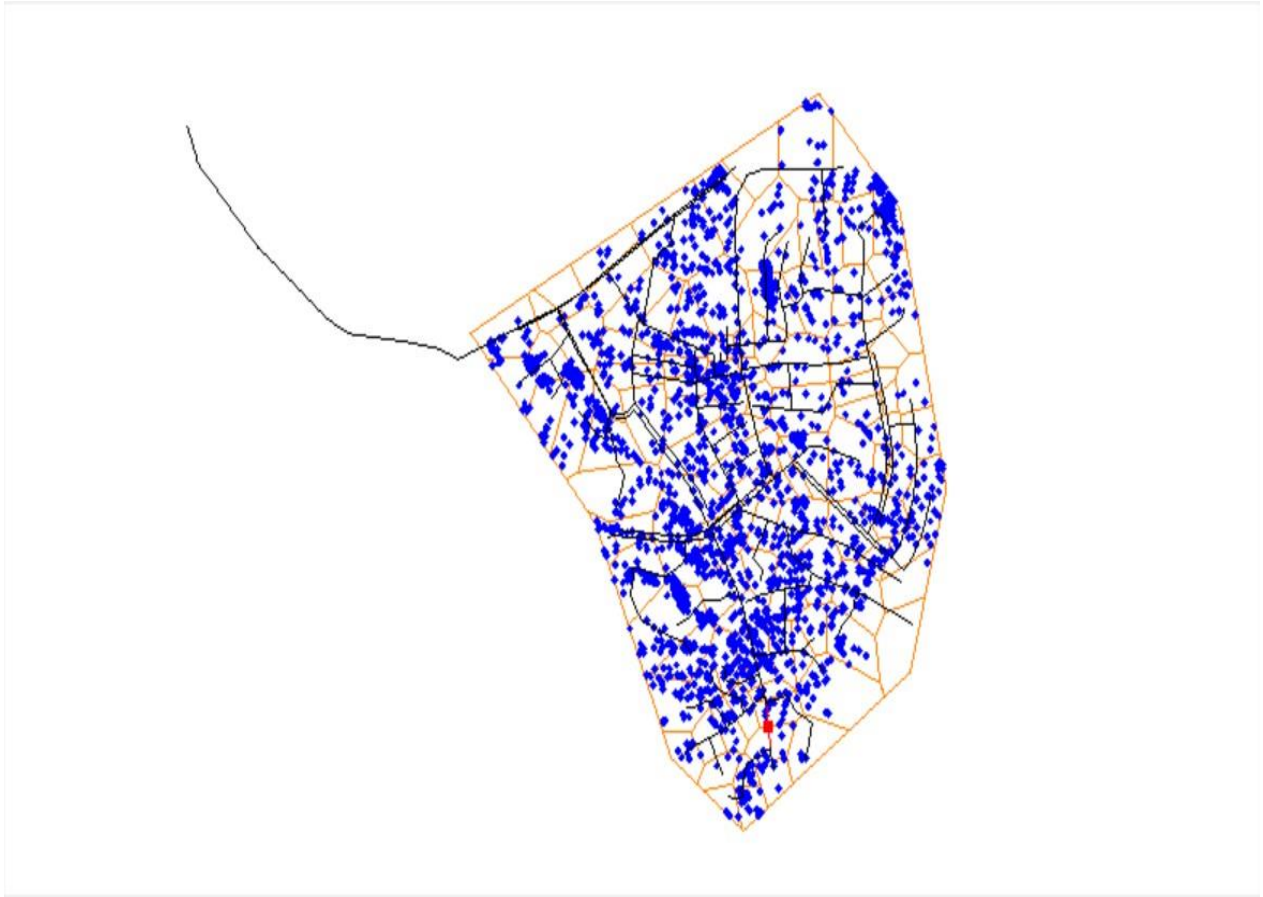


Figure 12 Thiessen polygons with water customers

water supplied is lost in the network which can be attributed to leakages in the system. However, the exact location of points of leakage was not known. To identify the leaking nodes, following approaches were used. To identify the leaking nodes, Darwin calibrator tool were used.

3.8 Leakage Detection using Darwin Calibrator tool

The supplied observed data is imported into the model and a Leakage Detection Run is performed with the help of the Darwin Calibrator tool to pinpoint possible nodes where water leaks can be expected.

The “Goodness of Fit” criteria used to assess the accuracy and model performance for this study is the Minimize Difference Squares. It aims to reduce the squared sum of the difference between the field data and the model results. The lower this number is, the better and reliable are the model results. The first step was to import the measured field data for flow and pressure data into the

model. In figure, the green dots represent real flow taken from the field, and the red line represent the model flow without leakage, it is only take consideration consumption (real) demands at the nodes. By using Darwin calibrator tool to try to simulate leakage at the nodes, by simulate leakage at the nodes to increase demand at the nodes, in order to match the real flow with the model flow by changing the emitter coefficient that represent leakage. 25 field data snapshot manually inserted in Darwin calibrator tool, each data set represent the field data at one of the time step over 24 hours. The field data was recorded every 60 minutes or 1 hours; it consists of pressure readings at three locations of the DMA as well as flow leaving the reservoir which enter to the DMA. The pressure was converted into hydraulic grade. the demand group were created with the nodes subjected to possible leakage.

For this study, all the nodes in the system were included due to the present uncertainty as to which nodes of the whole network present leakage. Once this step was completed, the first optimization run for leak detection within the water network was initiated. This first run was conducted during the period of low flow in the network (03:00), it is a time where the pressures in the network tend to be high and leaks in the network can be easily detected. The minimum, maximum and increment of the emitter coefficient were added as a criterion for the calibration run.

3.9 Model Calibration and Validation

The flow data used in this study consists of 2 datasets, one represents the simulated flow at the outlet pipe of the reservoir (P-123), which oscillates between 63.2 m³/h and 126.3 m³/h, and a second dataset of real measurements at the same pipe with values of discharge ranging between 91.6 m³/h and 183.1 m³/h. It is this discrepancy between both data sets that leads to the hypothesis of water leaks within the network and shows that the model needs to be calibrated.

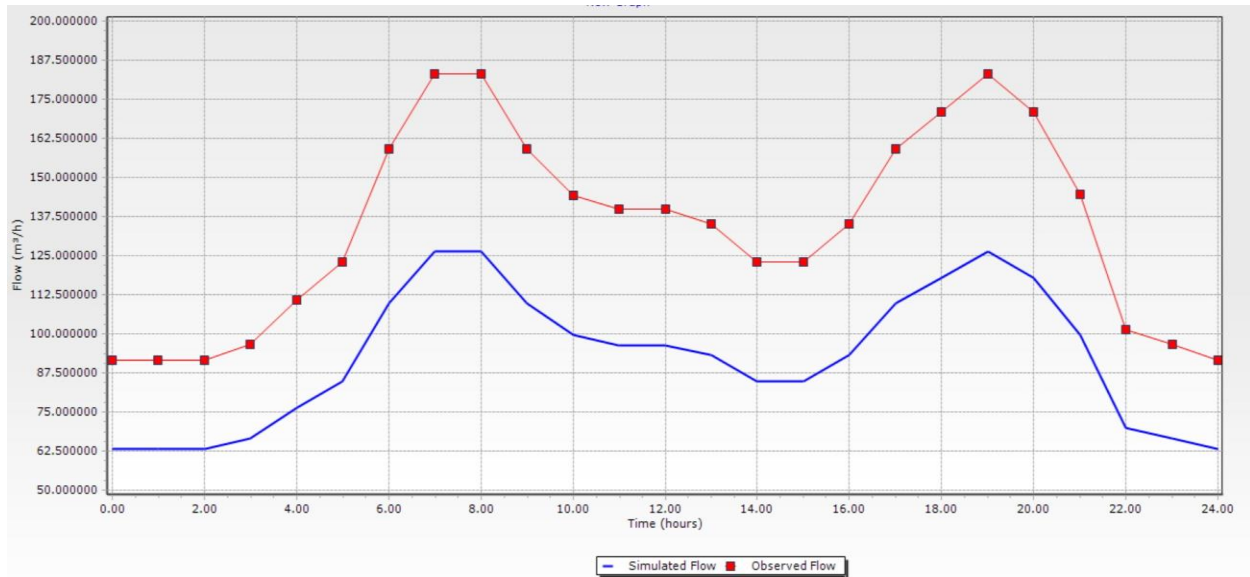


Figure 13 Flow Dataset

3.9.1 Model Calibration

Having estimated leakage rates using MNF method, the next step is model calibration. The aim of calibration is to minimize the differences between the observed performance and model predictions. The model calibration used in this study was Darwin Calibrator tool (Mohd et al., 2017).

3.9.1.1 Darwin Calibrator tool

Darwin Calibrator allows the user to calibrate a model either manually or, with efficient genetic algorithms, in a more automated fashion. It allows for multiple calibration candidates to be presented so the best possible solution to a given system can be found. Solutions can also be exported into a new scenario for use in an existing water system.

During the calibration process, pipe roughness coefficients and nodal demands were adjusted as calibration parameters while pressure and flow data was collected for model calibration. Although a large amount of “good” observation data is needed for estimating calibration parameters with sufficient confidence (Walski 2000), in this study data collection was limited to three points: inlet point, (Average Zonal Pressure) AZP and (Critical Point) CP due to time and financial constraints. In such circumstances, trade-offs between sample design costs, level of model accuracy and real-world constraints is necessary (Kapelan et al. 2003; Speight and Khanal 2009).

3.10 Water Distribution System Performance Evaluations

3.10.1 Methodology for Performance Indicators (PI) Definition, Selection, and Analysis

The Selection of performance indicators from the IWA/AWWA- PI and PIs for water loss assessment (WLA) developed by (H. E. Mutikanga, 2012). Performance Indicators for Water Supply Services: IWA Manual of Best Practice(Alegre H., Hirnir W., Baptista J., 2006) the number of indicators and variation on the existing indicators probably reached over one thousand, PI are grouped in a structure. The performance indicators were arranged in the following groups: water resources, personnel, physical, operational, quality of service, and financial. Each function can also have up to four levels of indicator, according to their importance as management tools. Performance indicators range from level 1 (basic), which provides a general management overview of efficiency and effectiveness, to level 3 (detailed) indicators, which deal with specific elements of operational management.

PIs for water loss assessment (WLA) proposed by (H. E. Mutikanga, 2012) was tested in 5 branch water utilities of NWSU- Uganda and extended to PI-based benchmarking in some African cities Dar-es-salaam (Tanzania), Nairobi (Kenya), Khartoum (Sudan), Lagos (Nigeria), Lilongwe (Malawi), Johannesburg (South Africa), Cape Town (South Africa), and Windhoek (Namibia). WLA PI contains 25 different performance indicators, the PIs were structured and arranged into six main groups operational (8 PIs), Asset Management (5 PIs), Meter Management (3 PIs), Illegal use management (3 PIs), Human resource management (3 PIs), and Economic and financial (3 PIs); The PI system input variables are structured in seven main domains, the same as those adopted in the IWA-PI A Water volume data, Personnel data, Physical assets data, Operational data, Customer data, Time data, and Economic and financial data totally 51 variables have been defined and a PI computational tool (H. E. Mutikanga, 2012).

Operational indicators (OpWL)

In this group, PIs are intended to assess the performance of the utility as regards operation and maintenance (O&M) activities. Utility managers need to pay much attention to O&M activities as the efficiency of the water utility can be lost or improved by these activities.

Asset serviceability indicators (AsWL)

In this group, PIs are intended to assess the long-term performance of the utility's assets to continue providing an acceptable level of service to its customers. There is normally a tendency for utilities to reduce asset renewing and operating cost to ensure short-term financial and economic sustainability especially in the developing countries.

Meter management indicators (MeWL)

In this group, PIs are intended to assess the utility's water meters functionality to accurately measure flows and safeguard against revenue losses. Choosing the right meters, keeping them in good operating mode or even the time to replace them presents big challenges to utilities in the developing countries and managerial efforts in this respect are assessed.

Illegal water use indicators (IWL)

In this group, PIs are intended to assess the efficiency of the utility in handling illegal use of water and managerial efforts to promote proactive illegal use investigations and control. Currently there is very limited use of performance indicators to effectively assess illegal use of water that have become endemic in most cities of developing countries.

Human resource (personnel) indicators (PeWL)

In this group, PIs are intended to assess the efficiency and productivity of the utility's human resources and managerial efforts in recruitment and training of staff.

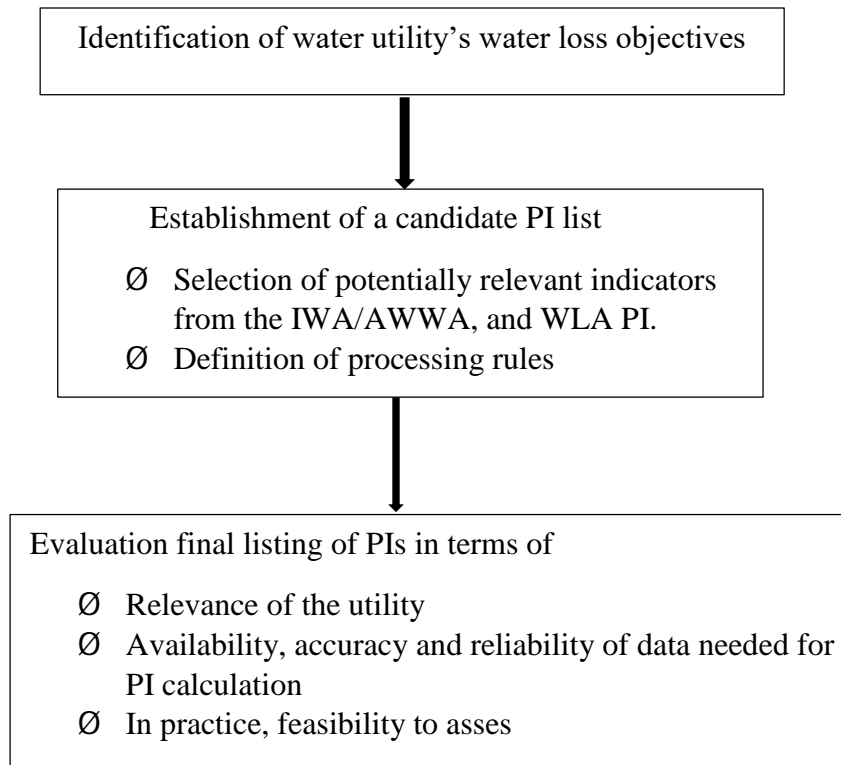


Figure 14 Methodology for PI Selection

The Criteria selected the Performance Indicators (PIs) for water loss Assessment in terms of

1. **Applicability to use** – No “one-size-fits-all” each utility must examine its own operational processes and practices, infrastructure, organizational culture, supporting data, technology and available resources and develop performance measures most appropriate for their local working environment. Performance Indicators (PIs) has their own basic Requirement or assumptions made in deriving/ calculating but not all are valid and practical for every water utility and some of PIs required detailed data that was difficult and costly to obtain (H. Mutikanga et al., 2010).
2. **Relevance to the utility**
3. **Availability, accuracy and reliability of data needed for PI calculation** - PIs are useless if data used to generate them is not reliable. The validity of the results, conclusions and decision-making are all based on the data quality.

4. **In practice, feasibility to assess** - The feasibility of the PIs was assessed using available data within the utility and additional easy to measure and less expensive data to collect but of much significance to water loss assessment (WLA).

Due to time and willingness to answer the selected PI was only used in three AAWSA branch water utilities to evaluate the performance of Addis Abeba's water distribution system. The three branches are Arada, Megenagna, Lemikura.

4 RESULTS AND DISCUSSIONS

4.1 Asses and quantify the water losses

4.1.1 Assessments of the apparent water loss components

In order to assess the apparent water loss components, metering accuracy, meter-reading errors, data handling and billing errors and illegal consumption were analysed. The results for each of these components of apparent loss are elaborated below

4.1.1.1 Water Loss due to Meter Failure

Using the procedure mentioned above, water used but unmeasured by DN 15 mm meters due to meter failures in Addis Abeba has been estimated and is summarized in Table 13.

Meter failure analysis

The total number of meter failures that were registered in the laboratory in the year 2023 was 39,476. Out of these, 29,896 were of size DN 15 mm. The number of meter failures was then used with the total number of meters (DN 15 mm) in the system to estimate the meter failure rate for the AAWDS for the DN 15mm size group.

Table 13 Estimates of Water Loss due to Meter Failure

Description	Unit	Value
Total service connections with meter sizes of DN 15 mm	No.	505,872
Connection efficiency (Active/Total)	%	90
Active service connections with meter sizes of 15 mm (MAC)	No.	455,284
Average water use for DN 15 mm service connection (Q)	M^3 /year	210
DN 15 mm meter failure rate (MFR)	Failures/meter/year	0.059
Detection-replacement/service time to failed meters (DRT)	Months	1
Estimated unregistered water due to failure (UWF)	M^3 /year	470,081
Unregistered water delivered per meter per year	M^3 /meter-year	0.93
System Input Volume (SIV)	M^3 /year	207,000,000
Non-revenue water (NRW)	M^3 /year	89,010,000
Non-revenue water (NRW)	%	43
% of SIV delivered but unregistered by DN 15 mm meters	%	0.228
% of NRW due to DN 15 mm meters	%	0.53

According to Table 13, 0.23 % of the total system input volume is lost as a result of the unmeasured water by DN 15 mm due to meter failure in Addis Abeba. In Addis Ababa, 0.53 % of the yearly NRW is attributed to meter failure for small customer meters (Table 13). The figures are comparable to those for Kampala, Uganda, which represent 1% of yearly NRW. (Table 13) shows 0.93 m³/meter-yr is the volume of water that an average meter cannot measure. Although the utility invests much in servicing defective meters, their influence on global NRW is limited, and more proactive techniques are necessary for cost-effective meter maintenance and long-term reduction of water losses.

$$\text{MFR} = \frac{\text{total number of meter failures}}{\text{total number of in-service meters}} = \frac{29,896}{505,872} = 0.059$$

The estimated meter failure rate (MFR) for the AAWDS (DN 15 mm) meters was 0.059 failures per meter per year. The total number of in-service meters (DN 15 mm) in AAWSA is 505,872 (82.2%)

However, not all reported defective meters ended up in the meter laboratory as records from the customer billing database indicated a meter reading with no change in reading for months till the meter reader reported as a defective meter. meters are also serviced on-site and information not sent to the meter laboratory for updating the database; thus the meter failure rate estimated is rather conservative. The problem with data from the billing database is that it does not lend itself to easy analysis. it does not provide meter failure data in cohorts of size, number, type, and cause of failure. It is a billing database not designed for water meter management. The meter failure was categorized according to type and cause of failure, as shown below, which was observed on field visit by, disassembling the meters, from laboratory records, and from interviews with a technical plumber and meter reader.

Observed Defects

- Ø Clogged strainer
- Ø Blocked of inlet strainer
- Ø Registered stuck
- Ø Misty glass
- Ø Worn-out impellers

According to the laboratory records, many meters failed due to clogged meters. This form of meter failure occurs in old and aged infrastructure old parts of the city of Addis Abeba like Arada, Addis Ketema, and Gulele sub cities; This defect is discovered when the flow indicator triangle does not rotate.

The other notable observed meter failure was that the inside part of the meter was melted (Figure 15). This might be due to a back flow of hot water from the shower or kitchen towards the water meter, or it could be due to people's reckless activity of starting a fire alongside the water meter to eliminate rubbish.



Figure 15 Shrinkage inside a water meter

Water meters are significantly affected by Suspended solids and depositions. Figure (16) shows Lime scale build-up in the inside of a water meter. A system with corroded pipes operating under intermittent conditions is likely to re-suspend a lot of deposited particulates in the pipe with devastating consequences for water quality and water meter performance (F. Arregui & Jr, 2005).



Figure 16 Lime Scale build-up in side of a water meter

The most common cause of meter failure is related to water quality. Due to high frequency of bursts and leaks coupled with poor pipe repair practices and inadequate mains flushing, a lot of silt and other suspended particulates enter the water system and get lodged in the meter drive mechanisms halting them from moving (H. E. Mutikanga, Sharma, et al., 2011a); (F. Arregui & Jr, 2005); and (Moahloli et al., 2019). In addition, meter movement is halted by deliberate meter vandalism by customers who would like to use water free of charge (H. E. Mutikanga, 2012).

Observed newer types of meter failure in laboratory this is probably due to intermittent supply, water quality, and water meter quality. Lund (1988) reported meter failures of 1% for newer types of meters in Seattle Water Department. The meter failure rate in Kampala is about 7 times that in Seattle despite advancement in metering technology over the years. This is probably due to the difference in their water supply systems characteristics. Research has shown that meters in developing countries are more likely to malfunction due to intermittent supply (Criminisi et al., 2009). Meter failure and accuracy are greatly influenced by the water supply system in which they are installed (H. E. Mutikanga, Sharma, et al., 2011b).

4.1.1.2 Water Loss due to Meter accuracy degradation rate

The comparative billing analysis

For all meter models, the meter accuracy degradation profile was developed, as shown in Figure 17-19. linear regression analysis was performed for each model, the following mathematical models were obtained for the degradation of Model 1, 2, and 3 meters according to Equation (3.2). The meter degradation models for Model 1, 2, and 3 meters and the coefficients of determination R^2 are shown in Equations (4.1)–(4.3), respectively.

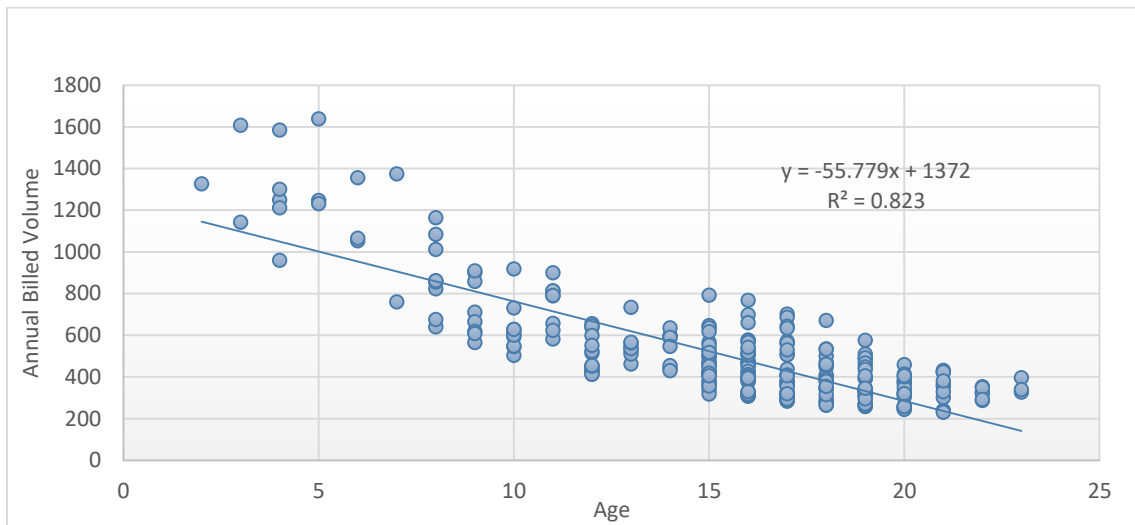


Figure 17 Meter degradation profile for model 1 meters

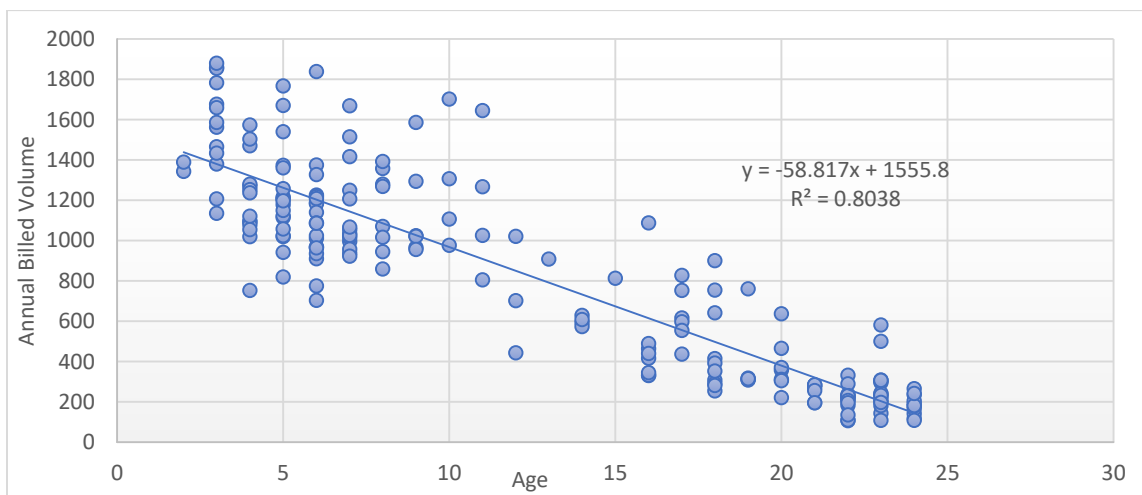


Figure 18 Meter degradation profile for model 2 meters

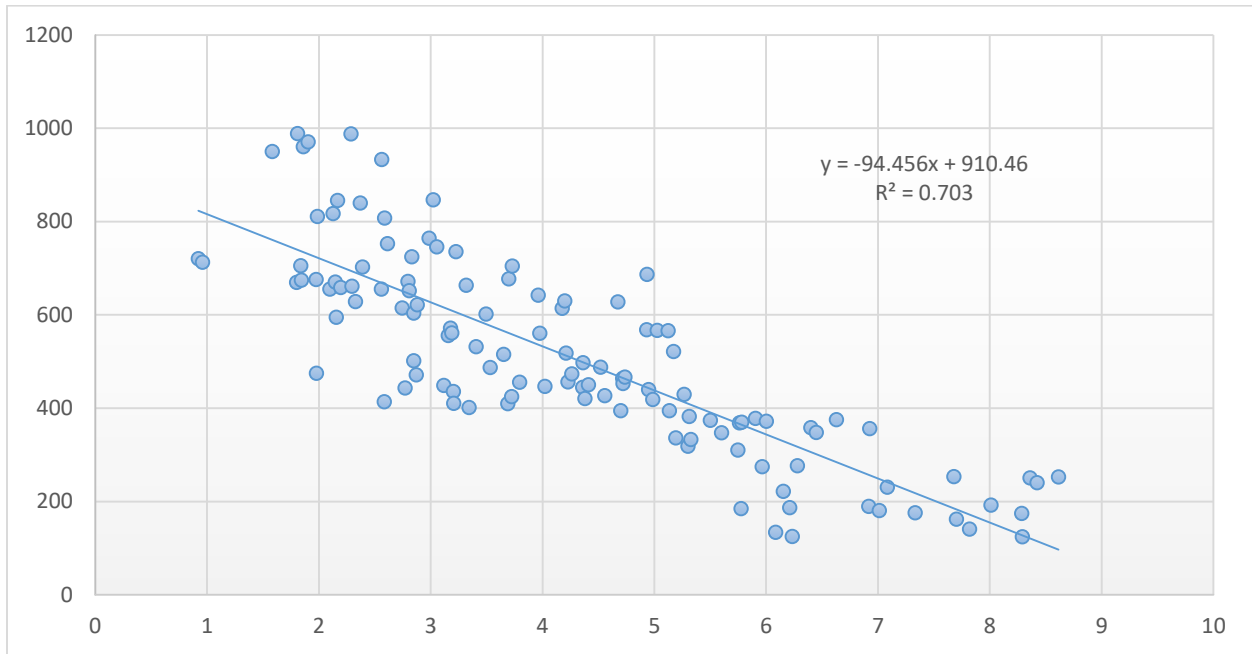


Figure 19 Meter degradation profile for model 3 meters

Model 1 meter:

$$y = -55.779x + 1372 , R^2 = 0.823 \dots \dots \dots (4.1)$$

Model 2 meter:

$$y = -58.817x + 1555.8 , R^2 = 0.8038 \dots \dots \dots (4.2)$$

Model 3 meter:

$$y = -94.456x + 910.46 , R^2 = 0.703 \dots \dots \dots (4.3)$$

where y is the annual billed volume (M^3) and x is the totalized age of the meter.

In Equations (4.1)– (4.3), the regression coefficients for all the meter models associated with meter age were generally negative, implying that the ability to register the volume through the meters depreciates over time. The coefficient of determination (R^2) shows the percentage of variation in y is explained by x. From the result of R^2 shown above, Age is determined the ability to register the volume through the meter. (Mbabazi et al., 2015) & (Musaazi et al., 2021) in a study reported

similar results on the effect of age on the metrological performance of two residential meter technologies are the volumetric meter and the multi jet-type velocity meter. (Mbabazi et al., 2015) in a study showed that the annual billed volume decreased with age influencing the degradation profiles for the meter models under-investigation.

The meter accuracy degradation rates were determined as 4.06 % per year for Model 1, 3.78 % per year for Model 2, and 10.4 % per year for Model 3 meters. The degradation rates obtained only considered meter age as the sole factor influencing meter degradation and did not include pressure, water quality, leaks, and users' storage tanks, and thus consideration of meter age does not explain meter degradation entirely. This was because considering all the factors involves rigorous meter and field tests which are expensive and cannot be afforded.

Model 3 had the highest degradation rate (10.4 % per year) when compared with all the other meter models. the degradation rate is also greater than the 2% per year degradation rate reported by (Couvelis & van Zyl, 2015) on the same size of meters (DN 15 mm) extracted from a water meter database in South Africa. No stratification into age groups was done in study.

Model 2 had the lowest degradation rate (3.78 % per year) when compared with all the other models. This finding is similar to results from a study by (Musaazi et al., 2021) which indicated that Model 2 (volumetric meters) were the best meters with the lowest meter accuracy degradation rates. However, the result differs from findings in a study by (Mbabazi et al., 2015) which showed that Model 3 (multi jet) meters had the lowest meter accuracy degradation rates. The difference could be attributed to variability in the calculation of meter age. (Mbabazi et al., 2015) determined the age from the time elapsed between the last two readings, date of field installation and date of last reading. This may not be applicable for an intermittent supply. Water meters, in particular volumetric type meters constituting 76% of the total installed meters in the KWDS, have been found to fail less than 5 months after installation due to water quality problems (H. E. Mutikanga, 2012).

With an average annual consumption of 210 m^3 , which was calculated from billing data, the amount of unmeasured water was determined by Equation (3.4) as 8.5, 7.9, and 21.84 m^3 per meter per year for Model 1, 2, and 3 meters, respectively. These values are relatively similar to the average annual loss due to meter inaccuracies of 22.8 m^3 per meter per year reported in the

Kampala Water balance report for the financial year 2011/2012. The values were less than the amount of unmeasured water of 58 m^3 per meter per year shown in the study (H. E. Mutikanga, 2012), because in their study losses were estimated due to metering errors and failures. The difference also shows an improvement in the management of non-revenue water at KWDS. The figures for the average annual unmeasured water were also less than the estimated figures for Westchester Joint Water Works in New York and Taunton Water Works (TWW) in Massachusetts, which were determined as $43 \text{ m}^3/\text{meter-yr}$ for both utilities in 1982 according to (H. E. Mutikanga, 2012), because losses were also estimated due to metering errors and failures. These annual losses obtained indicate that 4,071,546 m^3 of water are lost per year or (1.94 % of water lost from the total system input volume) due to meter inaccuracies of Model 1, Model 2, and Model 3 meters resulting in losses of US\$ 3 million per year for a water tariff with the average cost of ETB $6.2/\text{m}^3$ (US\$ $0.74/\text{m}^3$).

4.1.1.3 Meter Reading Error

Out of the total 6154 water meters audited to confirm accuracy of meter readings, only 92 water meters were found to have been incorrectly read. The meter reading error computed as a result was 1.5% of water sold.

4.1.1.4 Data handling and billing errors

A sample of 4774 consumer accounts was audited to confirm the accuracy of data input capture into the billing system. Only seven accounts were found with wrongly captured readings. The corresponding data-capturing error computed was 2.1 % of the water sold.

A sample of 74,000 customer accounts was audited for 6 months (September 2022 - March 2023) to confirm if there were Apparent losses from Data Analysis Errors in the Customer Billing System's archival and data reporting processes. A common source of error was inaccurate estimations utilized for accounts that did not yield a meter reading. It was also confirmed by a utility employee, although obtaining the actual number of client accounts proved challenging. Billing adjustments may provide clients with a legitimate monetary credit, but they do so by generating a negative value of consumption volume, so understating real consumption.

As a result of consumer complaints about incorrect billings, billing adjustments were made to 56 accounts. 652 m^3 of water were saved by actual visits on site by NRW staff workers from 72,035

billing accounts that showed zero consumption. Additionally, for 1829 new customer accounts without billing, it was unable to generate and send bills to the customers. Employees of utilities should check customer billing records on a regular basis to detect data problems and quantify these losses.

4.1.1.5 Assessment of Unauthorized Water use

Unauthorized consumption, including illegal connections and deliberate illegal use, is a significant issue leading to revenue loss and understating customer consumption. This study found there are two types of customers. Some customers make illegal connections without realizing it, such as using the same water meter for multiple compounds which is prohibited. Some customers deliberately use water illegally, such as drawing water from the system without paying for it.

Types of unauthorized water use found during this study:

- Ø Illegal connection by utility customers
- Ø Illegal connection by others
- Ø Changing the direction of water meter installation (“Robisho”)
- Ø Bypasses to customer consumption meters
- Ø Tampering with metering or meter reading equipment
- Ø Any other method of receiving water while preventing the water utility from collecting

4.1.2 Assessment of Leakage

4.1.2.1 Comparison of leakage estimation by different methods

The hydraulic modeling of water distribution system of the DMA was successfully simulated using Water GEMS. In this study various calibration runs were used to compare the model results with the observed flow. Each calibration run further improved the model results, reducing the Sum of Squared Errors (SSE), and making them match the measured flow on the main pipe.

This reduction of the SSE translates into a better fit between the observed flow and the model results as shown in the following figure:

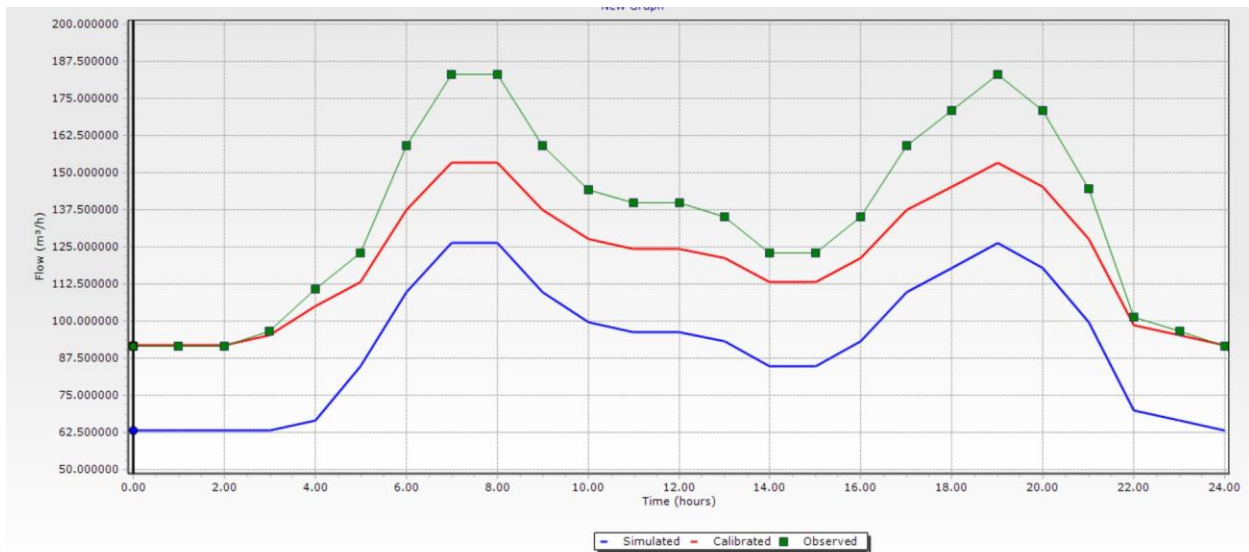


Figure 20 Comparison of simulated and observed flow into DMA

The Minimum Night Flow (MNF) methodology was applied to the real system to the DMA. Utilizing hourly pressure values monitored from Pressure Logger, the average Night-Day Factor Equation above for the three points was 24 h/day. The estimated real loss using MNF method was, $816 \text{ m}^3/\text{day}$.

As Darwin Calibrator simulated leakages, it could be seen that the Net Inflow in the system of DMA has increased from $63 \text{ m}^3/\text{h}$ to $92 \text{ m}^3/\text{h}$ when leakage is introduced by using Emitter Device. The model was able to simulate 45% leakage in the system. This is due to the fact that the DMA network system is old and has deteriorating pipes. This finding is consistent with the findings of other studies (Gajbhiye et al., 2017); (H. E. Mutikanga, Vairavamoorthy, et al., 2011). Leakage estimation using water GEMS ($29 \text{ m}^3/\text{h}$) and Minimum Night Flow method ($34 \text{ m}^3/\text{h}$) produced comparable findings.

Comparing both methods for estimating leakages in WDS, both approaches compared pretty well although the model gave a more conservative value of leakage and is likely to result in great advantages of water savings in practice than MNF analysis and water balancing approach. Because the network hydraulic modeling performed prior to calibration provides a global view of the entire network in terms of pressures, flow rates, water quality levels, consumption peaks, and reservoir levels, as well as several other state conditions. On the other hand, the Minimum Night Flow

method requires preliminary measurements at several points of a network prior to measurement itself (Cheung et al., 2010).

However, calibration model results may have some degree of uncertainty linked with field data and/or hydraulic WDS model. The ideal solution may differ physically from the genuine parameter values. Using an average leakage in both ways of $31.5 \text{ m}^3/\text{h}$ and projecting such loss over monthly periods, a volume of $21,995 \text{ m}^3$ of water was lost per month.

4.1.2.2 Location of the Leaks

Once the model was properly calibrated, a leak detection run was performed during the low flow hours of the model using data from 24 hours. After running a total of 12 simulations, the solutions started to converge to the nodes indicated on the map.

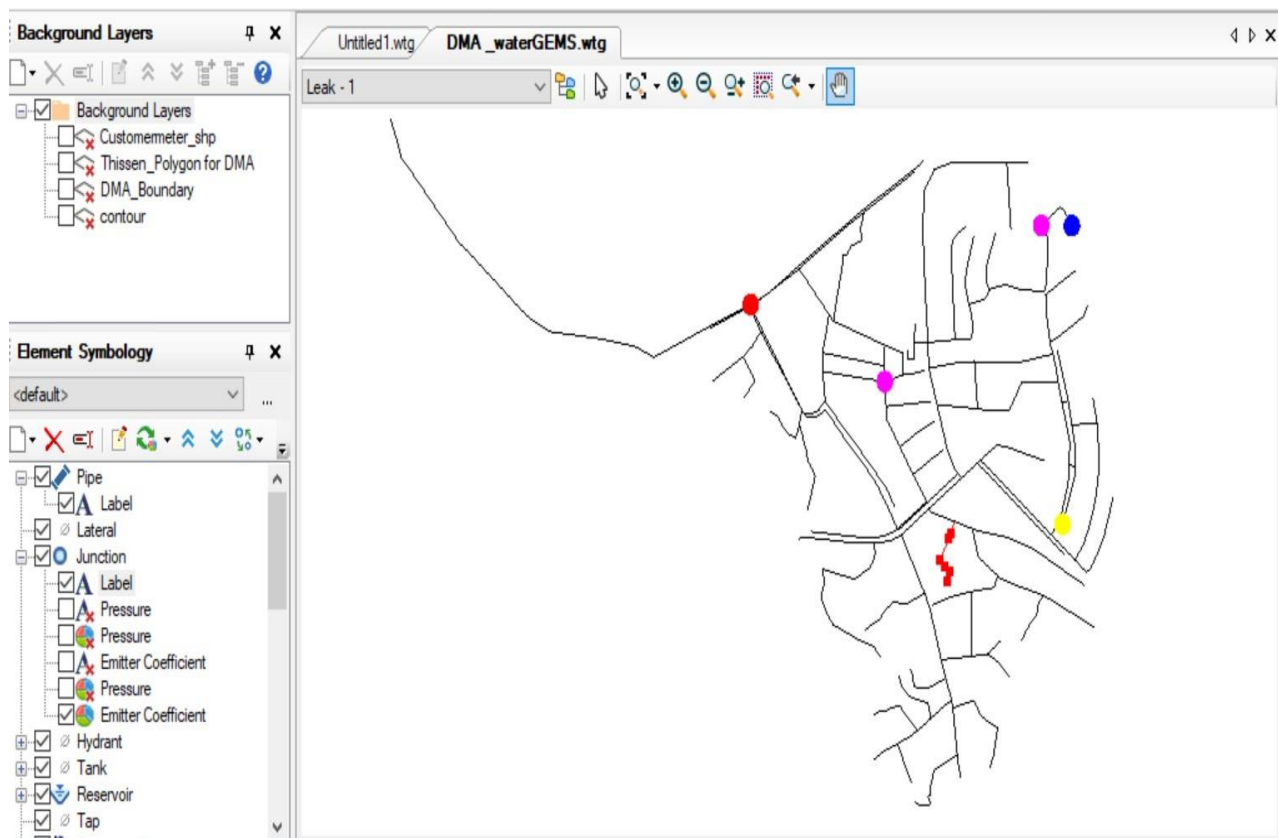


Figure 21 Locations of Nodes with Leakage

After calibration process, Darwin Calibrator generated a few nodes with Emitter Coefficients. The degree of magnitude of the Emitter Coefficient values, indicate that there are leaks with a very low discharge, that could have been difficult to identify without the help of the model.

Table 14 Obtained values of the Leakage Emitter Coefficient and Magnitude of Leakage

Node	Emitter Coefficient (L/s/(m H2O) ⁿ)	Leakage flow (m3/d)
J-96	0.36	512.4
J-108	0.06	80.16
J-24	0.03	42.96
J-121	0.03	40.56
J-89	0.01	14.4

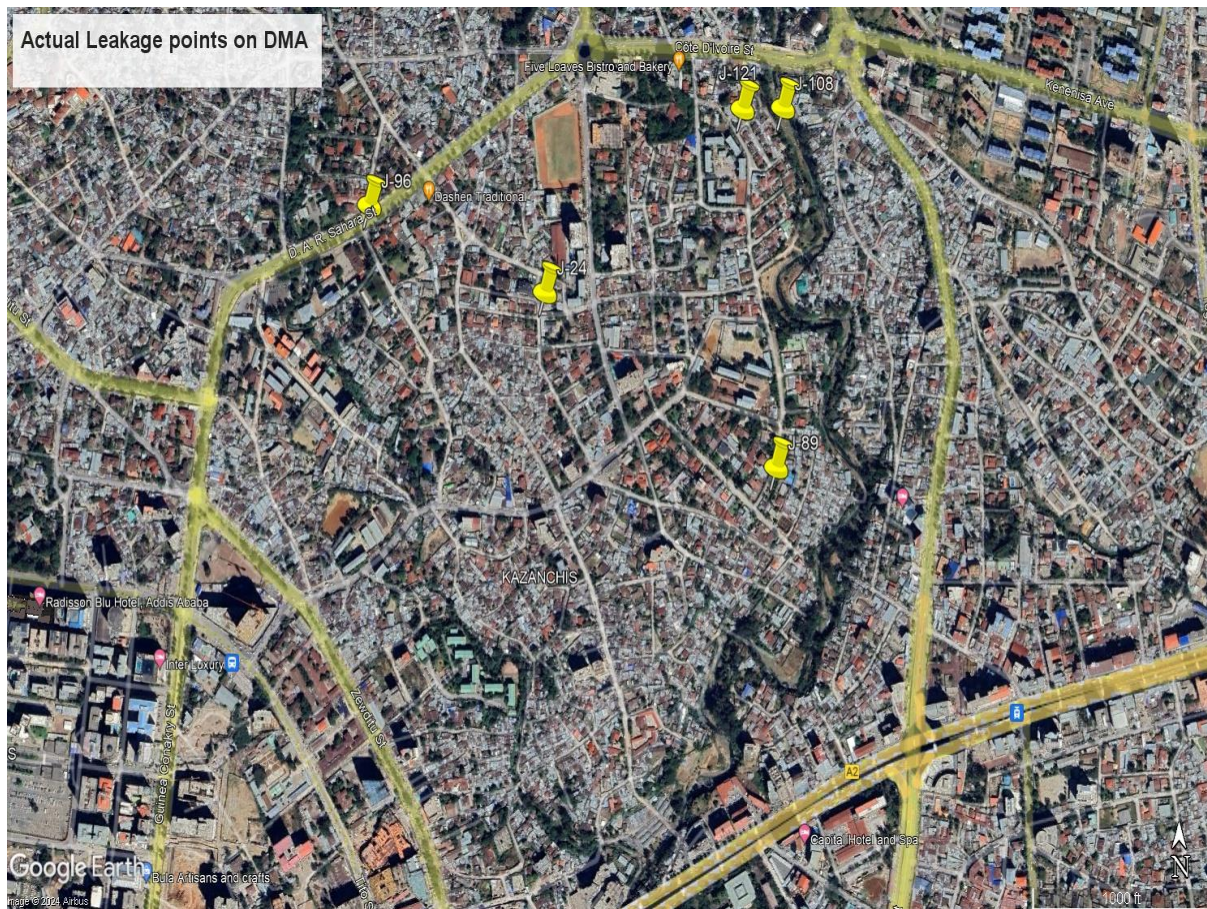


Figure 22 Leakage points on map

4.1.2.3 Model Validation

The model was verified for EPS and Figure 23 shows the simulated and observed pressure.

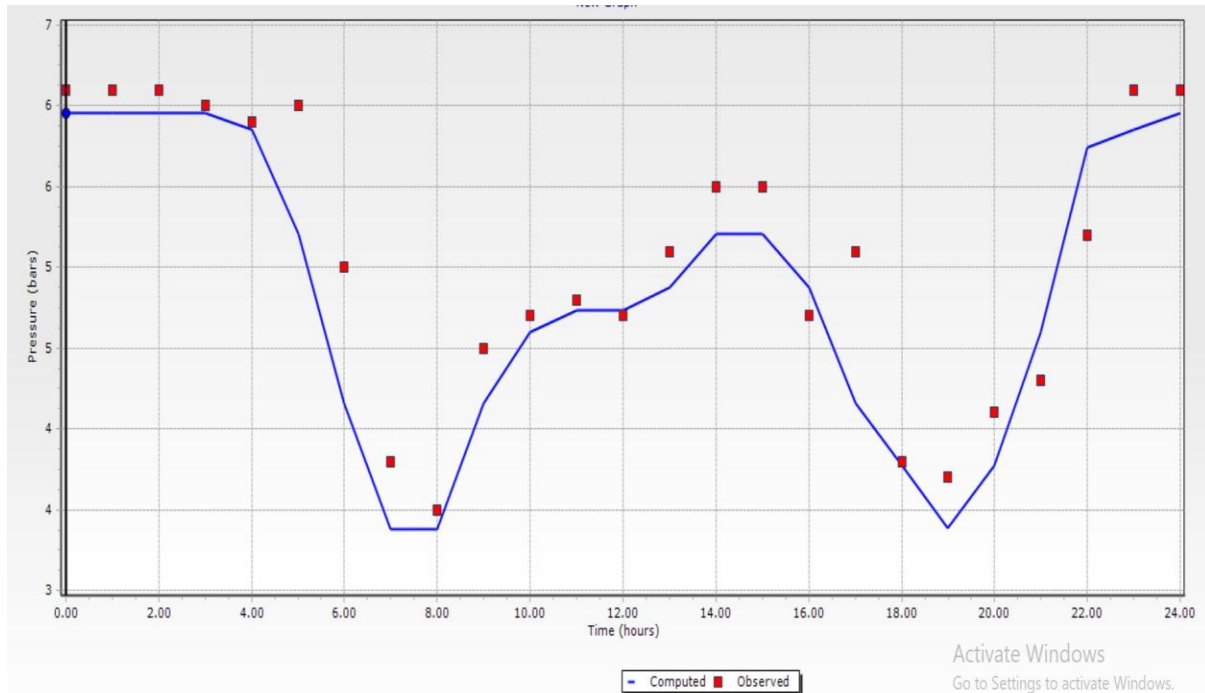


Figure 23 Comparison of computed and observed pressure

4.2 Water Distribution System Performance evaluations

4.2.1 Evaluating the effectiveness of the selected Performance Indicators (PIs)

In order to test the robustness of the selected PIs, a pilot implementation was carried out in three water utilities of AAWSA using preliminary data for 2022/2023 financial year. The feasibility of the PIs was assessed using available data within the utility. As a result, 3 PIs out of 12 proposed PIs were successfully tested. The objectives of the pilot testing were:

- Ø to confirm the relevance of the selected PIs;
- Ø to assess, in practice, the feasibility of assessment;
- Ø to reveal the information gap and any problems of data quality and availability
- Ø to do a preliminary performance benchmarking exercise and answer the question of: how well are we doing compared to others and how can we get better?

Table 15 Selected performance indicators for water loss assessment in AAWDS

From Operational indicators
OpWL1 – Leakage handling efficiency (%)
Concept: Number of reported leaks and bursts repaired within the target period / Total number of reported leaks and bursts
Processing rule: $OpWL1 = (D2 / D1) \times 100$
Variables: D1 = Total number of leaks and bursts reported during the assessment period
D2 = Total number of recorded leaks and bursts repaired within the target period
Objective of PI: To improve repair response time and minimize leakage run-times.
OpWL7 – NRW per connection (m3 /connection/year)
Concept: (NRW during the assessment period x 365/assessment period) / number of service connections
Processing rule: $OpWL7 = (A14 \times 365 / F1) / C4$
Variables: A14 = Non-revenue water (m3) C4 = Service connections (No.) F1 = Assessment period (day)
Objective of PI: To compare performance among different utilities.
Op27 – Real losses per connection (l/connection/day when system is pressurized)
Concept: Real losses during the assessment period x 1000 / (number of service connections x number of hours system is pressurized during the assessment period / 24)
Processing rule: $Op27 = A19 \times 1000 / (C24 \times H2 / 24)$
Variables: A19 - Real losses (m3)
C24 - Service connections (No.)
H2 - Time system is pressurized (hour)
Objective of PI: To quantify amount of Real Losses that do not generate revenue.
OpWL8 – Transport availability (No. / 100 km)
Concept: Number of vehicles, motor cycles and bicycles available daily, on a permanent basis, in average, for field works in operations and maintenance activities / total mains length x 100
Processing rule: $OpWL8 = D15 / C1 \times 100$

Variables: C1 = Mains length (km) D15 = Permanent transport (vehicles, motor cycles and bicycles) (No.)
Objective of PI: To assess speed capacity of utility regarding repairing of failures and for comparisons with other undertakings.
OpWL5 - Apparent losses ratio (-)
Concept: [Apparent losses during the assessment period / Total amount of billed metered authorized consumption (including exported water) during the assessment period] x 100
Processing rule: $OpWL5 = (A11 / A3) \times 100$
Variables: A11 = Apparent losses (m ³) A3 = Authorized billed metered consumption (m ³)
Objective of PI: To quantify amount of commercial losses that do not generate revenue out of authorized consumption
For Asset Serviceability Indicators
As WL 1 – Mains failures (No./100 km/year)
Concept: (Number of mains failures during the assessment period (including failures of valves and fittings) x 365 / assessment period) / total mains length x 100
Processing Rule: $As WL1 = (D28 \times 365 / H1) / C8 \times 100$
Variables: C8 - Mains length (km)
D28 - Mains failures (No.)
H1 - Assessment period (day)
Objective of PI: To assess the long-term performance of the utility's assets to continue providing an acceptable level of service to its customers.
As WL2 - Service connection failures (No./1000 connections/year)
Concept: (Number of service connection failures during the assessment period x 365 / assessment period) / number of service connections x 1000
Processing Rule: $As WL2 = (D29 \times 365 / H1) / C24 \times 1000$
Variables: C24 - Service connections (No.)
D29 - Service connection failures (No.)
H1 - Assessment period (day)
Objective of PI: To assess the long-term performance of the utility's assets to continue providing an acceptable level of service to its customers.

As WL3 - Water-point failures (No./water-point/year)
Concept: Number of water-points failures during the reference period / total number of water points
Processing Rules: $As\ WL3 = (D32 \times 365 / H1) / F6$
Variables: D32 - Water-point failures (No.)
F6 - Water-points (No.)
H1 - Assessment period (day)
Objective of PI: To Assess the impact of Water-point failures on level of service.
Comment: Water point refers to public taps and standpipes. Water-point failures shall be accounted for regardless of their cause (tap, pump, piping, water source, etc.).
For Meter Management indicators
MeWL14 – Meter reading efficiency (%)
Concept: (Number of complying meter readings / total meter reading audits carried out during the assessment period) x 100
Processing rule: $MeWL14 = (D9 / D8) \times 100$
Variables: D8 = Meter reading audits (No.)
D9 = Complying meter readings (No.)
Objective of PI: To minimize apparent loses due to meter reading errors
MeWL15 –Meter Failure (%/year)
Concept: (Number of customer water meters that are reported defective (stuck) during the assessment period
Processing rule: $MeWL15 = (D10 / E1) \times 100$
Variables: D10 = Meter Failure (No.)
E1 = Customer meters (No.)
Objective of PI: To assess utility’s level of defective meters (stuck meters) at reference time.
For Illegal Water use Indicators
IIWL17 – Illegal use detection efficiency (%)
Concept: (Number of illegal cases confirmed / total number of service connections investigated during the assessment period) x 100
Processing rule: $IIWL17 = (D13 / D12) \times 100$

Variables: D12 = illegal use cases reported and investigated (No.)
D13 = illegal use cases confirmed (No.)
Objective of PI: To assess managerial efforts to proactively manage illegal water use.
IIWL18 – Fines recovery efficiency (%)
Concept: (Number of illegal use fines recovered / total number fines levied during the assessment period) x 100
Processing rule: $IIWL18 = (G2 / G1) \times 100$
Variables: G1 = Illegal fines levied (No.)
G2 = Illegal fines paid (No.)
Objective of PI: To assess managerial efforts in enforcing laws and regulations to fight illegal use of water and improve financial sustainability.
For Human Resource (Personnel) Indicators
PeWL20 – Operations and maintenance staff (No. / 100 km)
Concept: Number of full time equivalent employees working in operations and maintenance of the water transmission, storage and distribution system / total mains length x 100
Processing rule: $PeWL20 = B1 / C1 \times 100$
Variables: B1 = Operation and Maintenance personnel (No.)
C1 = Mains length (km)
Objective of PI: To assess the efficiency human resources deployment in handling water losses.
Comments: Essential when with high employee levels but poorly deployed for effective management of water losses. Useful for benchmarking. The indicator is assessed for a reference date.

The results of the Performance Indicators based Water Loss assessment using some of the proposed PIs are presented in Table 15 for AAWSA branch utilities

Table 15 PI - based water loss assessment in AAWSA branch utility

Branch Utility	No. of Service Connection	NRW (m ³ /Conn/day)	Leakage Handling Efficiency (%)	Service connection failures (No./1000 connections/year)
Arada	49,000	0.55	40	65
Lemikura	65,654	0.2	46	41.67
Megenagna	39,857	0.29	44	53.9

The following observations can be made from Table 15

Leakage handling efficiency (LHE)

The average value observed for LHE was 43.3 % with a minimum value of 40% and a higher value of 46%, which means that most reported visible leaks take more than a day before they are repaired. The utility practices more of reactive maintenance than preventive maintenance. There is potential for improvement in branch utilities with respect to speed and quality of repairing leaks to bring it at par with other well performing utilities in other countries like Uganda utility, Entebbe and Mbale (H. E. Mutikanga, 2012). In general, leaks control should be reviewed and more proactive strategies adopted to minimize leak run-times and associated costs.

Asset Management

The mains and service lines replacements are far from ideal for AAWSA distribution networks which are over 40 years old. From the year of installation data extracted from the Auto CAD file. The existing AAWSA distribution system is 3,381km long with sizes ranging from 50mm up to 450mm composed of different pipe materials including DCI, GS, HDPE, ISOPE, ISOPVC and Steel pipes and nearly 50% of AAWSA distribution pipes are of uPVC materials. Also about 80% of the distribution network is below 100mm diameter. Most of the distribution line are functioning beyond their life span. Considering the existing distribution networks are over 50 years old; it can

be said that the branch utilities doing few replacements at all despite the high number of service connection failures and leakage level. Table 15 shown that service connection failures for Arada, Lemikura, and Megenagna are (65 No./1000 connections/year), (41.67 No./1000 connections/year), and (53.9 No./1000 connections/year) respectively. The high number of mains and service failures indicate poor structural performance of the water distribution systems in AAWSA. These network deficiencies cause frequent service interruptions and consequent customer complaints.

Non-Revenue water (NRW)

Out of nine branch utilities of AAWSA, only three utilities were able to provide data for the basic NRW indicator (m³ /connection/day) due to time and data limitation. Aging of pipe is one of the reasons for increased water loss in the distribution network and could be the major factor that contribute to the high NRW for the utility. From the findings depicted in Table 15 NRW was identified as 0.2 m³/connection/day. Which is high when compared to that of Uganda water utilities 0.05 m³/connection/day (H. E. Mutikanga, 2012).

4.2.2 Applicability of The Infrastructure Leakage Index (ILI)

The Infrastructure Leakage Index (ILI) is a measure of how well a distribution network is managed (maintained, repaired, and rehabilitated) for the control of real losses, at the current operating pressure. It is defined as the ratio of current annual volume real losses (CARL) to the unavoidable annual real losses (UARL) (Lambert et al. 1999). For well managed WDSs, ILI is equal to one or very close to 1 and tends to increase as the system grows older. Infrastructural leakage index (ILI) is proposed PI for real losses, and the one mostly used PI in developed countries.

During the review, The proposed PI for real losses, Infrastructure Leakage Index (ILI) (Lambert et al., 1999) & (Alegre H., Hirnir W., Baptista J., 2006), it was found inappropriate for use in Addis Abeba Water distribution system (AAWDS) for the following reasons. (1) it required detailed data that was difficult and costly to obtain. (2) no financial constraints to effectively undertake active leakage control and reduce leakage to the least technically possible levels.

The UARL, which is the lowest technically achievable level of leakage that could be achieved at current operating pressures assuming (Lambert et al., 1999):

- Ø the infrastructure is in good condition;
- Ø intensive state-of-the-art active leakage control, and
- Ø all detectable leaks and bursts are repaired quickly and efficiently.

All assumption made in deriving the unavoidable annual real losses (UARL) are not valid and practical for AAWDS for the following reasons.

- Ø The assumptions made for the empirical formula of UARL do break down for WDSs with financial constraints and where leakage control is reactive other than proactive. From the result shown in section 4.6 for Leakage Handling Efficiency for branch utilities most reported visible leaks take more than a day before they are repaired. Where, UARL only measures performance on active leakage control- ‘‘ find and fix’’ leakage activities and excludes efforts made using other leakage reduction options (Lambert et al., 1999).
- Ø From the year of installation date that has been extracted from the Auto CAD file obtained; The existing AAWSA water distribution network is over 40 years this implies that mains and service line replacement is far from ideal due to that high number of mains and service connections failure recorded as mentioned from the result found in section 4.6 the infrastructure is not in good condition where, UARL only calculated when the infrastructure is in good condition (Lambert et al., 1999).
- Ø For the UARL calculation, the coefficient 0.8 L/service connection/day/ m pressure used in the equation was based on one service connection to one customer. In real practice, this not happened as one service connection could serve more than one customer e.g. a block of apartments in Arabsa, yeka Abado, Jemo condominium. Generally, the assumptions used for estimating components of ILI (CARL and UARL) have not been universally accepted.
- Ø It does not recognize benefits of Pressure Management (PM) as default values are calibrated at a reference pressure of 50m. ILI will not change by lowering or increasing system pressure;
- Ø The ILI of 0.3 reported in the Netherlands and Austria indicates that UARL is greater than CARL, proving the ILI deficits even in developed countries' water systems.

From the fact that recent results of ILI assessments from around the world in a study carried out on 197 water undertakings around the world by the IWA water loss task force team,

ILI figures varied from 0.3 (Austria and Netherlands) to 598 in South East Asia have indicated that it is not suitable for comparing systems from different countries, especially developed countries with well-managed systems and developing countries with intermittent supply and low pressure systems (H. Mutikanga et al., 2010). They suggested ILI target values of 2 for well-managed systems in the developed countries and 5 for developing countries. The analysis of the data set for 30 water utilities in South Africa (Seago et al., 2004), indicates poor correlation between NRW and ILI probably due to the fact that ILI is an indicator of leakage in the system as opposed to NRW that includes apparent losses and unbilled authorized consumption on top of leakage.

4.3 Possible Causes of the Water Loss

The significant amount of water loss in Addis Abeba city can be attributed to numerous problems, which are discussed below.

Causes of Real Losses

Water losses occur in every water distribution network (WDN) in the world. For economic and technical reasons, it has to be accepted that real water losses cannot be entirely eliminated. Nevertheless, there has been a large increase in the knowledge and development of state-of-the-art equipment, allowing us to manage water losses within economic limits. According to the WHO leakage management manual (2001), four key factors influence the degree of leakage within a utility's pipe network. These are;

- Ø Availability of water, financial and personnel resources
- Ø Infrastructural conditions regarding materials, system pressure and renewal policy
- Ø Leakage control policy: activity, perception, technical expertise
- Ø Institutional attitude with respect to structure, regulation, politics

Age of Pipe Networks

The likelihood of pipe damage grows with age. Pipe leakage could be mostly caused by its age. However, not all pipelines will eventually fail (Sambodja et al., 2020). Ageing of pipes is one of the problems in AAWDS and found to be one of the major causes for increased leakage. Totally

there are over 580km of pipes aged above 40 years which are serving beyond their life span and needing replacement. According to the information extracted from the Auto Cad file, the transmission mains are over 50 years old, while the distribution pipelines are on average of 40 years old, making them highly prone to leaking. Due to the ages of the pipe networks, there have been frequent pipe bursts and leaks in the "inner city" areas like Arada, Gulele, and Merkato, which consist of older infrastructure. This information was acquired from discussion with the staff and during the field visit.

Causes of Apparent losses

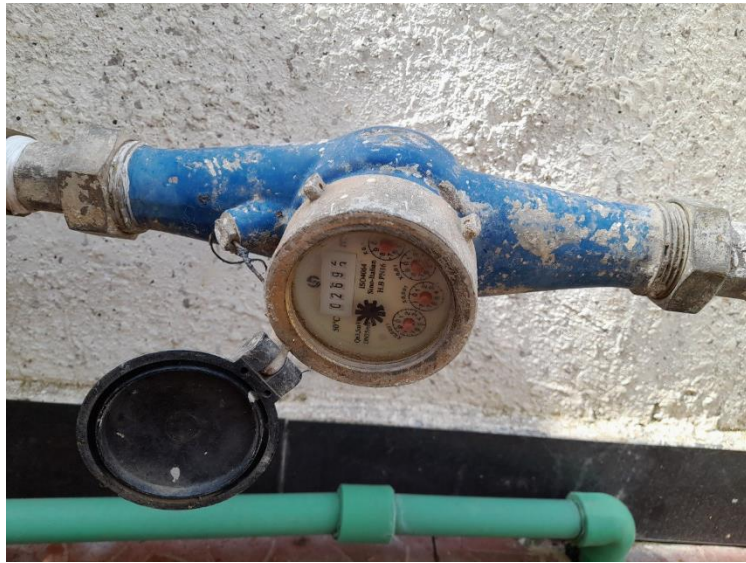
Meter inaccuracies

Water meters are imperfect tools, much like any other measurement tool. Every water meter, regardless of technology, has specific measuring constraints that prevent it from recording the exact amount of water used by customers when it is installed (Criminisi et al., 2009) & (H. E. Mutikanga, Sharma, et al., 2011a). In the case of AAWSA +/- 2% (+2 over registration and -2 under registration), water meter errors are acceptable. The user is frequently not paid for water consumed since it is not documented. According to local experts, until recently, the water authorities did not examine consumer meters unless they requested a checkup. When over registration occurs. Then, an in-service water meter was removed and tested in the laboratory to determine the cause. In either case, it is critical to quantify the size of these measuring errors, as meter inaccuracies are acknowledged as a key contributor to apparent losses.

The following are major influence elements that affect the accuracy both for domestic and industrial water meters based on their technology.

Incorrect meter installation

Meters are designed to be mounted horizontally, and any other orientation influences their performance (F. Arregui & Jr, 2005). If the installation is improper, the frictional resistance between the water meter's wing wheel shafts will increase, reducing the sensitivity of the water meter and eventually resulting in inaccurate measurement data. The more inclined the water meter is installed, the slower it will run. The effect is only noticeable at low flow rates (F. Arregui & Jr, 2005). During field visits in different parts of the water networks, erroneous meter installations



with varying mounting positions were observed (15°, 45°, 90°, etc.) which could lead to water loss.



Figure 23 Incorrect Meter Installation

The study for a water utility in one of the islands of Indonesia identified, the effect of incorrect meter installation could lead to Apparent loss of roughly around 20% and the rate wear and tear of the meter would be faster than normal due to the friction between the impeller and pivot spindle which will damage the measuring components in the long run

Another important consideration about the mounting position is that if it is not correct, according to the manufacturer recommendations, it may lead to a higher degradation rate of the meter. The reason is that in such cases the moving parts or the meter are not working and standing properly

Suspended solids and depositions

These two parameters have a substantial impact on water meters. Specially the water distribution networks have a water quality problem. According to the information gathered from utility employees, the water meter found in AAWDS has affected by suspended solid and deposition like inlet strainer blockage. Depositions may cause over registration, at medium-high flows and under registration at low flows. However, on the long term, depositions grow so large that they can prevent the impeller from rotating, temporarily or permanently, causing a severe under registration of the meter.



Figure 24 Partial blockage of inlet strainer

Illegal Connection

Water illegally removed from fire hydrants, illegal connections, bypasses to customer consumption meters, or tampering with metering or meter reading equipment; and any other method of receiving water while preventing the water utility from collecting revenue for the water. Unauthorized consumption results in unrecognized money and a mistake that underestimates customer usage. In most water utilities, this volume is low. In Addis Abeba City, there are a substantial number of

illegal water consumers within the distribution system. When there was a site inspection, the information acquired from utility workers was used. There were customers who were using one water meter for two distinct compounds without realizing it was prohibited. Customer intentionally makes an unauthorized connection and bypasses.

4.4 Possible intervention method to reduce water losses

water loss reduction requires a holistic approach: Increased leak detection activities alone will not solve the problem if infrastructure conditions deteriorate at the same time. Furthermore, even if financial resources are available, they will not have a positive effect unless the water utility has adequate structures and takes a proactive approach towards water loss management.

Pressure management can be an immediate and cost-effective solution for decreasing real water losses in a distribution network, even at low initial pressures. It also offers water conservation benefits because some types of water consumption will decline due to the reduced average zone pressure, for example from taps, showers and garden irrigation systems. A study by the IWA Water Loss Task Force found that pressure reduction results in a significant decrease in new pipe breaks and bursts (Fallis et al., 2011). Further benefits include deferred replacements and the extended service life of pipes, joints and fittings as well as fewer customer plumbing and appliance problems. All of these positive effects of pressure management usually result in high water savings and thus have very short payback times.

5 CONCLUSION AND RECOMMENDATION

5.1 Conclusion

This study assesses and quantifying total water losses in Addis Abeba water distribution system (AAWDS) by disaggregating real and apparent losses in order to develop appropriate strategies for performance improvement and evaluating the efficiency of the distribution system by selecting relevant performance indicators (PIs) for water loss assessment using the proposed participatory methodology for establishing a performance assessment system based on IWA/AWWA-PI concept and the developed computational tool (WLA-PI) for developing countries. The major findings of the study can be summarized as follows:

- Ø Metering inaccuracies, meter reading errors, data handling and billing errors, and illegal water consumption all cause apparent losses in AAWDS. The findings revealed meter reading errors (1.5% of water sold), data handling and billing errors (2.1% of water sold).
- Ø Addis Abeba water distribution system has a high meter failure rate (5.9% / year) for DN 15 mm water meters. Meter failure for small customer meters in Addis Abeba makes up about 0.53% of annually NRW, with the amount of water unmeasured by an average meter is 0.93 m³/meter-yr. The findings indicated that the meter failure was caused by the use of inappropriate metering technology, insufficient system operation and maintenance, particularly poor repair practices, irregular supply, and insufficient rehabilitation measures. The observed defects included a clogged strainer, a blocked inlet strainer, registered stuck, misty glass, and worn-out impellers.
- Ø Meter accuracies affected by meter type, manufacturer, and age of the meter.
- Ø Meter accuracy degradation rates for different meter models for the size of DN 15 mm were determined and calculates annual water losses due to meter inaccuracies. The result indicated that meter accuracy degradation rates were 4.06% per year for Model 1, 3.78% per year for Model 2, and 10.4% per year for Model 3 meters. Annual water losses due to meter inaccuracies were 8.5 m³ for Model 1, 7.9 m³ for Model 2, and 21.84 m³ for Model 3, totaling to US\$1.5 million per year.
- Ø Network hydraulic modeling and calibration are fundamental steps for efficient management of water losses in AAWDS. The results reveal that a calibration approach was used for modeling leakage and leakage detection in one real-world case study (DMA

discovered in the Kasanchis Area of Addis Abeba). It was discovered that water GEMs v10 can model leakage using an Emitter device.

- Ø The leakage was estimated using water GEMs (29 m³/h) and the Minimum Night Flow technique (34 m³/h). Using an average leakage rate of (31.5 m³/h) for both ways and projecting such loss to daily periods, a volume of 756 m³ of water is lost per day. This amount could serve 6300 individuals per day, accounting for 52.2% of DMA's entire population.
- Ø The Calibration Technique is an interesting alternative to be implemented in utilities when comparing both investigated methods for estimating leakages in AAWDS, because the network hydraulic modeling performed prior to calibration allows for a global visualization of the entire network, including pressures, flow rates, water quality levels, consumption peaks, reservoir levels, and several other state conditions. On the other hand, the MNF approach involves preliminary measurements in numerous sites of a network.
- Ø Identifying leaks in the study area's water distribution system led to the shortlisting of 5 leak spots. The model was able to simulate 45% leakage in the system, which is the real life situation.
- Ø Assessments of water distribution system efficiency using performance indicators (PIs) were shown to be viable. A total of 12 PIs have been suggested based on the IWA-PI concept and the WLA-PI tool developed for developing countries, some of which being successfully tested on AAWSA's three branch utilities. The remaining PIs have not been analyzed due to the time required to collect data. The utility's performance was effectively evaluated by the tested PIs in terms of NRW, asset management, and leakage handling efficiency. It was determined that the PIs were suitable for assessing the efficiency of water distribution systems. It is suitable for benchmarking water utility performance and for reporting "in accordance" with the IWA Guidelines.
- Ø The major assumptions used in estimating the Infrastructure Leakage Index (ILI) components—specifically, the UARL—do not suit local conditions, operational procedures and practices, infrastructure, organizational culture, supporting data, technology, and available resources. As a result, the ILI was found inappropriate for use in the AAWSA.

5.2 Recommendation

In light of the key findings of this study, the following recommendations are proposed:

- Ø AAWSA needs to develop an integrated water meter management system (IWMM) in order to reduce revenue losses brought on by metering inaccuracies and failures. The framework contains guidelines for the optimal meter size and selection, as well as the optimal time to replace meters.
- Ø As meter accuracy degradation rates will offer a more reliable estimate of when to replace meters, this study suggests using degradation rates to determine the ideal meter replacement period. AAWSA should also take this into consideration.
- Ø More proactive approaches from AAWSA are required for long-term, cost-effective water loss reduction and meter maintenance.
- Ø Further research on metering errors should be conducted, including removing the in-service water meter and testing it in a laboratory. To determine the impact of metering errors on water loss.
- Ø AAWSA consider using ILI in future as resources become available and technology evolves in line with IWA's recommendation of a step-by-step implementation.
- Ø To reduce illegal water consumption, revenue protection units (Illegal Water Consumption Investigators) must be establish. AAWSA should also take this into consideration.
- Ø AAWSA must consider advertising a cash prize for anyone with information on illegal water consumption to report to the utility and, if proven, a cash reward for the individual.
- Ø Network hydraulic modeling and calibration techniques should be taken into consideration by AAWSA to efficiently manage water loss in WDS.
- Ø AAWSA should implement performance indicator - based water loss management.
- Ø The volumetric water meter is the most suitable meter that is suggested for the AAWSA because of its low accuracy degradation rate.

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