MODELLING OF WATER LOSS IN HYDRAULIC DISTRIBUTION SYSTEM OF MINNA, NIGER STATE, NIGERIA

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A THESIS SUBMITTED TO THE POSTGRADUATE SCHOOL FEDERAL UNIVERSITY OF TECHNOLOGY, MINNA, NIGERIA, IN PARTIAL FULFILMENT OF THE REQUIRENTS FOR THE AWARD OF THE DEGREE OF DOCTOR OF PHILOSOPHY (PhD) IN CIVIL ENGINEERING (WATER RESOURCES AND ENVIRONMENTAL ENGINEERING OPTION)

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ABSTRACT

The main objective of this study is to model water loss in the distribution network of Shiroro District Metered Area, Minna, Niger State capital which has been facing severe water supply scarcity in recent time. This scarcity has been further complicated with high water loses via leakages. Hence, there is need for new and modern approaches which will involve increased automation for estimation of water leakages. The hydraulic machine, EPANET was used for the hydraulic modelling of the water loss in the networks of Minna Water Treatment Plant and Shiroro District Metered Area, (DMA). Active Leakage Control mechanism was adopted by deploying the use of modern leak detecting equipment. Leaks were collected and measured from 37 nodes of Shiroro distribution network. 24 hours Extended Period Simulation, (EPS), was carried out from leak prone nodes by varying emitter coefficient from 0.1, 0.15, 0.2 and 0.3 respectively. The leakage, Q_{leaks} generated from the model using discharge coefficient and the values of observed leaks $Q_{measured}$ from site were compared statistically using Nash-Sutcliffe efficiency model to check the performance of the model. Data set collated were further utilised in neural networks for the estimation of leakages. The leak generated using modern leak detecting equipment was $1.3m^3$ for eight (8) hours which was large because it was unnoticed. Pearson Product Correlation of pressure and leak was found to be very high positive and statistically significant. The NSE values obtained at optimum leakage coefficient of 0.2 for 8th hour, 9th hour, 10th hour and 11th hour are 0.73, 0.68, 0.58 and 0.52 respectively. These values are good indictors of good performance. Model validation for 20 pm, 21 pm and 22 pm hours in M. I. Wushishi Distribution Network gave NASH values of 0.767, 0.668 and 0.601 respectively. For validation procedure using ANN, for estimation of leakage at M. I. Wushishi Network, R^2 was 0. 65 and relative errors for training and testing were 0.367 and 0.215 respectively. R^2 of 0.65 is an indication of better performance of the model.in terms of leakage estimation. The performance of the model has suggested that using the emitter coefficient of 0.2 can model the water leakage in Shiroro and M. I. Wushsishi Water Distribution systems because of high correlation between the simulated and the observed discharges. Variation of leakage exponent on the performance of emitter equation in estimating leakages in water distribution system is recommended.

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ABBREVIATIONS

ALC	Active Leakage Control
ARIMA	Auto-Regression Integration Moving Average
ANN	Artificial Neural Network
AWWA	American Water Works Association
BABE	Burst and Background Estimates
CARL	Current Annual Real Loss
DMA	District Metered Area
DN	Diameter Nominal
EpaCAD	EPANET CAD
EPANET	Environmental Protection Agency Network Software
EPS	Extended Period Simulation
EWT	Elevated Water tank
FAVAD	Fixed Area Variable Area Discharge
ICF	Infrastructure Correction Factor
ILI	Infrastructure Leakage Index
IWA	International Water Association
MLP	Multi-Layer Perception
MNF	Minimum Night Flow
MRA	Multiple Regression Analysis
NRW	Non-Revenue Water
NSE	NASH Sutcliffe Efficiency Coefficient Model
OFWAT	Office of Water Services (UK)
RMSE	Root Mean Square Error
SPA	Single Period Analyses
SPSS	Statistical Package for Social Sciences
UARL	Unavoidable Annual Real Loss
UK	United Kingdom
USAID	United State Agency for International Development

WBI	World Bank Institute
WDS	Water Distribution System
WHO	World Health Organization
WRC	Water research Commission
WTP	Water Treatment Plant

CHAPTER ONE

1.0 INTRODUCTION

1.1 Background to the Study

Water management is the control and movement of water resources to minimize damage to life and property and to maximize efficient beneficial use. Many developing countries are facing crisis like drought and lack of access to safe drinking water in water management due to increasing population, water scarcity, equitable water distribution, water contaminations and effects of world economic crises (Cosgrove and Rijsberman, 2000). The water loss management scenarios in developed countries differ from those in developing countries in many ways. The main difference is in the response strategies and response sensitivity of the water utilities and governments of these countries.

Utilities in developed countries have managed to reduce their water losses to fairly acceptable and manageable ranges. In Greece's Larissa city, Non-Revenue Water, (NRW) has been estimated at 34 % of the System Input Volume, (SIV) (Kanakoudis and Tsitsifli, 2010), while between 15 and 60 % NRW is lost in Italy (Fantozzi and Fantozzi, 2008). Furthermore, Portugal loses between 20 and 50 % in NRW (Marques and Monteiro, 2003). The Netherlands have reported leakages of 3 % to 7 % of the water distribution input (Beuken *et al.*, 2006). The USA has an average NRW of 15 %, ranging from 7.5 % to 20 % (Beecher, 2002). In the UK, about 20 % to 23 % of water delivered is lost through leakage (OFWAT, 2010). According to Carpenter *et al*. (2003), the NRW levels for Australia ranges from 9.5 to 22 %, with a mean value of 13.8 %. On the other hand, leakage levels for Ontario, Canada, ranges from 7 to 34 % of the SIV. In as much as the developed countries have made significant efforts in reducing water losses, there is room for improvement. The wide variation in water losses in one country implies inconsistency in the way water losses are managed within the same country. One of the major issues affecting water utilities in the developing world is the considerable difference between the amount of water put into the distribution system and the amount supplied to consumers, the difference is called “Non-Revenue Water”

Current statistical surveys indicated that (NRW) in developing countries is around 45 to 50 % that is half of the total system input volume. The statistical procedure performs Multiple Regression Analysis, (MRA) with various physical and operational parameters as independent variables and the (NRW) ratio as a dependent variable. MRA is needed to select the most influential parameters of the ratio, and can also be used for estimating the ratio (Dongwoo and Gyewoon, 2017). High levels of water losses are indicative of poor governance and poor physical condition of the Water Distribution System, (WDS), (Mamlook and Al-Jayyous, 2003). The amount of water loss in water distribution systems varies widely from one system to another, from as low as 3 to 7 % to as high as 50 % of distribution input volume in the well-maintained systems of developed countries and less maintained system in developing countries respectively (Liemberger *et al.*, 2002). The (WDSs) in the Netherlands are probably the most efficient in the world as low leakage levels in the range of 3 to 7 % of system input volume have been reported (Beuken *et al.*, 2006).

Regular maintenance of infrastructure also helps to maintain water efficiency levels and is more cost-effective than rehabilitation (Makaya, 2014). Many water distribution systems in developing countries are operated under intermittent conditions (WWAP, 2014). As a result, water supply efficiency in these countries is compromised. Besides the problems associated with intermittent water supply conditions, water losses in developing countries have reached alarming rates, with non-revenue water levels in excess of 50% having been recorded in many of these countries. The slow progress in water loss reduction in developing countries is characterised by political interferences and institutional resistance to change (Gumbo and Van der Zaag, 2002).

One of the main causes of slow progress is that the utilities and water supply companies have not been ploughing back proceeds for network rehabilitation. Most of the revenue collected by water operators is diverted to other uses instead of maintaining and upgrading water distribution systems. In Latin American water utilities (NRW) levels of 40 % to 55 % are documented, of which Brazil accounts for 39% of the (SIV). On the other hand NRW ranging 44 % in Singapore to 63.8 % in Maynilad, Manila have been reported (ADB, 2010) while 50 % to 65 % of the NRW is due to apparent losses (McIntosh, 2003). In Africa NRW figures ranging from 25% in some South African towns to 70 % in Liberia have been reported (WSP, 2009). Zimbabwe has recorded (NRW) of up to 60 % of the SIV. (City of Harare, 2011). The upper limit in developing countries signifies the severity of the water loss problem. Hence developing countries should seriously consider reducing their water losses in order to operate sustainably. Non-revenue water has stood out as one of the major challenges faced by water utilities. Although there are signs of knowledge of (NRW) by utility management, the problem seems to be continually haunting water utilities. The problem seems to be less understood by water managers since little attention is paid at mitigating both physical and apparent water losses.

Each year more than 32 billion cubic metres of treated water is lost through leakages from the distribution networks. An additional 16 billion cubic metres per year is delivered to consumers but not invoiced because of theft, poor metering or illegal use (Simbeye, 2010). Such water losses would amount to about US$14 billion yearly. By avoiding such water losses, in excess of 100 million consumers would be serviced without new capital outlays (World Bank, 2006). The loss of treated water in the distribution system results in direct loss of revenue for the Public Water System, (PWS). In the USA it is estimated that there are close to 237,600 pipe bursts per year translating to 2.8 billion dollars lost in yearly revenue (AWWA, 2003). Water operators and service providers in developing countries suffer from poor strategic management, weak financial and operational management, unskilled staff, low funding priority, weak customer service orientation and political interference.

Furthermore, few or no standards exist in many developing countries for evaluating performance with respect to non-revenue water (Marin, 2009). Thus, it can be inferred that the challenges faced by water utilities in developing countries are centred on water leakage losses, and as such water loss management should be made a key priority, with water leakage management on the top list. Many funding organisations, including the World Bank and Africa Development Bank, have made efforts to reduce non-revenue water in their projects which have included the following: prioritizing water loss reduction, inclusion of (NRW) reduction components, and setting targets for reduced NRW as a condition for funding. Faced with a host of water loss management challenges, developing countries are making efforts to reduce water losses in their water distribution systems. In Africa, and particularly in South Africa, the Water Research Commission (WRC) developed various software for understanding and reducing non-revenue water. The initiative came about when the South African Water Research Commission (WRC) discovered that the reduction of non-revenue water was one of the key problem issues facing the continent (McKenzie, 2002). To assist water suppliers in addressing their water losses, the commission developed a suite of models and associated documentation. The models currently available are:

i) The SANFLOW minimum night flow analysis model.

ii) The PRESMAC pressure management model.

iii) The ECONOLEAK active leakage control assessment model.

iv) The AQUALITE water balance model.

1.2 Statement of the Research Problem

Water loss occurs in Water Distribution Systems (WDS). It reflects the ability of utility to manage its network. A high level of real loss reduces the amount of precious water reaching consumers, increases the operating costs of the utility and makes capital investments in new resource schemes larger. A high level of apparent losses reduces the principal revenue stream to the utility. Segmasin Nig Ltd (1998) reported that losses in water distribution system in some urban areas in Nigeria is as high as 50 %. In order to reduce water losses and improve efficiency of delivering water to customers, the condition of the (WDS) needs to be very well understood and decision-makers (DMs) need to solve the problem of how much water is being lost, where and why? Although direct real time assessment methods such as in-line inspections are ideal, their high costs practically limit their application in most water utilities of the developing countries. In such cases, indirect performance measures such as the water balance and Performance Indicators (PIs) should be considered. Whereas a range of performance assessment and water loss control manuals are available, (Alegre *et al.,* 2006 ; AWWA, 2009). Thus, providing a good foundation for water loss reduction, the tools and methods proposed therein do not fully address the unique characteristics of (WDSs) in developing countries and therefore cannot be directly applied. In addition, the most widely used indicator for water distribution efficiency is percentage (NRW).

This (PI) is misleading as it is heavily influenced by consumption which has nothing to do with the condition and operation of the (WDS). Another problem is that most (WDS) performance measures widely used in the developed countries such as the unavoidable annual real losses (UARL) and the infrastructure index (ILI) (Lambert *et al.,* 1999), are dubious in the context of most developing countries with financial constraints to effectively undertake active leakage control and reduce leakage to the least technically possible levels. Sustainable water loss control is a complex problem with economic, environmental, social and public health dimensions. Although various water loss reduction strategies do exist, deciding on which option to choose amidst often conflicting multiple objectives and different interests of stakeholders is a challenging task for water utility managers. This is further complicated in developing countries with either imperfect data or lack of it. Clearly, knowledge gaps still exist with respect to water loss control in water distribution systems particularly in the developing countries. This research seeks to bridge the knowledge gaps by developing appropriate methodologies for estimating water losses in not so well managed water distribution systems of the study area, A simple methodology was presented by (Gianfredi *et al.*, 2016) by which the analyses of inflow data records were collected in several water distribution networks. Leakages were assessed based on the seasonal fluctuation of water consumptions. The methodology was tested on two synthetic case studies based on the Apulian region WDN in Italy, where hydraulic status was simulated by an advanced WDN model that included a realistic pressure-dependent background leakage model.

The resulting estimates of leakages proved to be accurate under the analysed condition. (Soldevila *et al.,* 2019), have presented leak localization method using Kriging Interpolation in which the node with the highest difference in current and non-leak pressure values were identified as the leak nodes

1.3 Aim and Objectives

The aim of this study is to model loss via leakages in water distribution system in Minna.

The specific objectives are to:

i. evaluate domestic water supply of Minna
ii. identify positions of leakages in water distribution system using leak detecting techniques
iii. investigate the relationship between losses and pressure distribution in water distribution system, and
iv. estimate the water loss in water distribution system using the model developed

1.4 Justification for the Study

The modelling of water loss in hydraulic distribution system will be of immense importance in reducing Non-Revenue Water which will translate to saving millions of Naira and ensuring water supply with adequate pressure to consumers. The model to be developed could be applicable to other urban systems in the country. For developing countries, reducing water losses by half would avail over 22 million m^3/d which would be enough water to serve over 100 million people and water utilities would be able to recoup about US $ 3 billion every year that could be used to improve service coverage particularly for the urban poor (Kingdom *et al.,* 2006). These figures highlight the importance of the research with the aim of solving the water loss problem in urban (WDSs) of the developing countries.

1.5 Scope and Limitation of the Study

The scope of the study is the overall modelling of water loss via leakage in the Shiroro District Metered Areas, (DMA), in Minna Water Distribution System. The modelling was narrowed to Shiroro DMA which has thirty-seven (37) nodes as against 892 nodes in the entire network due to computational cost of the training and field observations of all eight pressure zones at the same times of hours of 8, 9, 10, 11, 20, 21 and 22.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 Water Demand Management

Water demand is defined as the volume of water required by users to satisfy their needs. In a simplified way it is often considered equal to water consumption, although conceptually the two terms do not have the same meaning Wallingford (2003). In most developing countries, the theoretical water demand considerably exceeds the actual consumptive water use. Water demand management refers to any socially beneficial action that reduces average or peak water withdrawals or consumption form either surface or ground water, consistent with the protection or enhancement of water quality (Tate, 2000). According to Rothert (2000), water demand management is the adaptation and implementation of a strategy by a institution to influence the water demand and usage in order to meet any of the following objectives: economic efficiency, social development, social equity (Mwendera *et al.*, 2003).

Urban water demand is usually quoted in terms of litre per capital per day (1/capita/day). Despite the variation in residential indoor water use from household to household, a typical pattern (referred to as the water use profile) can be developed to provide a reasonable representation of indoor water use, based on the different indoor water use components (kitchen, bathroom, laundry, and toilet) and household occupancy (Mitchell *et al.*, 2000). In many African cities urban water demands are often homogeneous owing to a range of levels of services occurring within the same urban area. Levels of service can vary from household connections to standpipes or to no service at all Wolday (2005). Water demand for industrial, agricultural, fire service and others (losses and public use) were expressed as a percentage of domestic demand, 40%. These percentages were estimated by Agunwamba (2008).

2.2 Methods of Measuring and Comparing Water Losses

2.2.1 Measuring water losses

The Non-Revenue Water (NRW) is expressed as percentage of the total consumption. The the Minimum Night Flows (MNF) per connection are the most commonly used methods of measuring losses. NRW is the measure of losses over a period as the difference between the amount of water put in to a system and the metered or estimated quantity of water taken by consumers, while (MNF) is an indicator of the probable rate of losses at a given time. Night flow measured in moderately sized sectors (up to around 3000 service connections) are extremely useful for identifying the presence of existing unreported leaks and bursts, and the occurrence of new ones. However, continuous night flows can also be used for assessing annual average real losses (Farley, 2003). NRW is a useful indicator of probable losses, but it may overestimate them because supply meters tend to under-record consumption. In UK, figures for (NRW) tend to be unreliable because the un-metered consumptions have to be estimated and can be up to 10 % in error.

Attempts to compare the performance of different undertakings by measuring some uniform figure for domestic consumption can be misleading. Many factors influence NRW and differ from one undertaking to another, standards of housing, rates of occupancy, age of mains, length of mains per 1000 population served proportion of trade and bulk supplies, ground condition, etc. (Twort *et al.*, 1994). The minimum night flow (MNF) per property connection is a better indicator of loss rates in part of a system. However, figures of this type are affected by the characteristics of an area; in dense urban areas there will be more blocks of flats with large storages which may fill at night. Nevertheless, the MNF is a good direct indicator of the state of parts of a system (Twort *et al.*, 1994). On the other hand, Warren (2005) referring to fully metered situations, considers that the annual water balance can initially only be taken as a guide as the calculations are susceptible to errors, analyses show this uncertainty in the calculated annual losses to be +/-4 6 %.

Different countries use different methodologies to evaluate the losses like the U.K. leakage practitioners and planners consider leakage almost exclusively in terms of night flow rates, rather than as a calculation of annual losses as in West Germany, (IWA, 2003). Each method has its respective merits. 'Annual losses' are used for retrospective assessment of overall performance and long-term demand forecasting. 'Night flows' are used by practitioners responsible for leakage control and prioritization of leakage control activities. Any conceptual model therefore needs to be able to link night flows with annual losses in a consistent manner. Although percentage figure is rarely meaningful when comparing different organizations, they can be used to indicate the extents of reduction of water loss by a single water supplier (WHO, 2001).

2.2.1.1 Estimating water loss from discovered leaks

Losses from leaks that are discovered should be measured to determine the rate of loss and the total volume lost during the life of the leak. Three methods are suggested by (AWWA, 1992) in Leak Detection Productivity document

1. Bucket and stop watch method.
2. Hose and meter technique.
3. Modified orifice and friction formula. method

The first method involves holding a container against the leak for a period of time and recording the time it takes to fill the container to a predetermined level. The leak is expressed as the equation of flow per unit time. The second method requires connecting a hose to the leak and directing the flow through a meter.to a container to determine the volume at a given time. The third method is the simplest to perform in the field but requires calculation. This method is often helpful for large leaks where the flow is too great to measure. It requires that the size and shape of the hole be measured and the line pressure be determined. A pressure gauge or a hand-held blade pedometer could be used to determine the pressure of the water coming from the leak or a nearby fire hydrant. This method also uses some assumptions regarding the shape of the hole that may introduce error. For losses from such items as a pipe or broken taps, Douglas Greeley equation assumes an orifice coefficient of 0.80 and calculates flow in gallons per minute from the formula:

$$Q = (43767)/1400 \times A \times \sqrt{p} \tag{2.1}$$

estimation of leak from transmission mains

Where,

Q= flow in gallons per minute

A= the cross-sectional area of the leak in square inches and

P= the pressure in pounds per square inch.

A leak loss for circular holes under different pressure is estimated by Douglas S. Greeley formula as

$$Q = (30{,}394) \times A \times \sqrt{p} \tag{2.2}$$

estimation of leaks from circular holes

where

A is the cross-sectional area of the leak in square inches and p is the pressure in pounds per square inch.

2.2.1.2 Leaks losses for joints and cracks under different pressure

For leaks emitted from joints and cracked service pipes an orifice coefficient of 0.60 is used in the following equation

$$Q = (22.796) \times A \times \sqrt{p} \tag{2.3}$$

leaks from Cracks and Joints Where, "A" is the area in square inches and "P" is the pressure in pounds per square inch.

2.2.2 Water balance approach

The amount of water loss differs from country to country, city-to-city and even from network to another network within one city. Different countries use different indicators to evaluate their status in comparison with other and to compare the distribution system in order to take action based on the level of losses. As stated above comparison using UFW expressed as a percentage has limitation when used for comparison as it highly depends with the volume of the water produced. The traditional performance indicators of water losses are frequently expressed as a percentage of input volume.

However, this indicator fails to take account of any of the main local influences. Consequently, it cannot be considered to be an appropriate performance indicator (PI) for comparisons (WHO, 2001). Depending upon the consumption per service connection, the same volume of real losses/services connection/day, in percentage terms, is anything from 44 % to 2.4 % Thus countries with relatively low consumption like the developing countries, can appear to have high losses when expressed in percentage terms, in contrast percentage losses for urban areas in developed countries with high consumption can be equally misleading (Farley, 2003). To account for the wide diversity of definitions related to water loss, many practitioners have identified an urgent need for a common international terminology that among them task forces from the international water association (IWA) recently produced a standard approach for water balance calculation with a definition of all terms involved as indicated in Figure 2.1

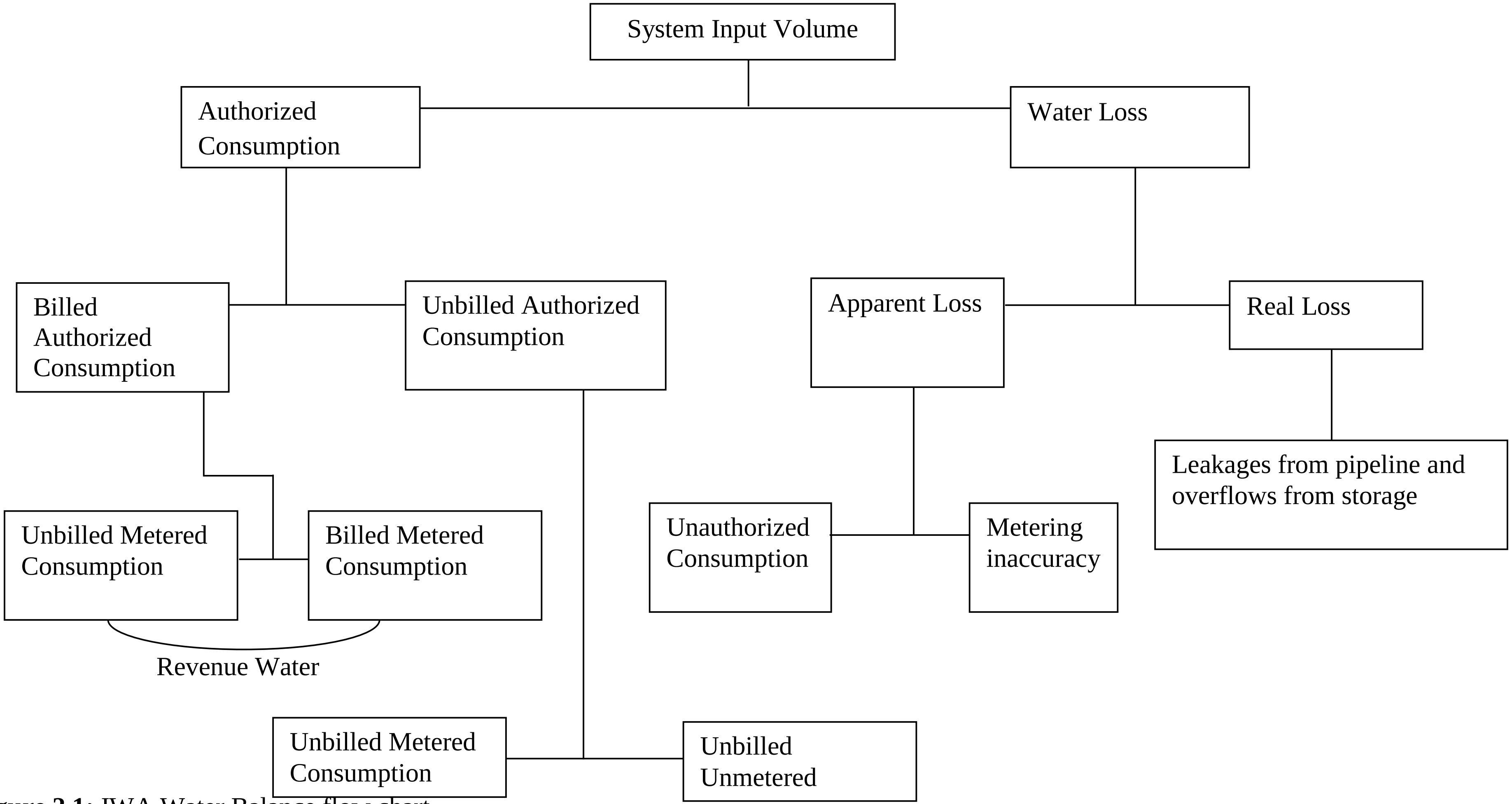


Figure 2.1: IWA Water Balance flow chart

Source: Farley and Trow (2003)

1. System input volume is the annual volume input to that part of the water supply system
2. Authorized consumption is the annual volume of metered and/or non metered water taken by registered customers, the water supplier and other who are explicitly or implicitly authorized to do so. It includes water exported, and leaks and overflows after the point of customer metering.
3. Non-revenue water (NRW) is the difference between system input volumes and billed authorized consumption
4. Water losses are the difference between systems in put volume and authorize consumption, and consist of apparent losses and real losses.
5. Apparent losses consist of unauthorized consumption and all types of metering in accuracies
6. Real losses are the annual volumes lost through all types of leaks, bursts and over flows on mains service reservoirs and service connection up to the point customer metering. `   Accordingly, the quantity in Figure 2.1 can quantified as explained in the following steps (Liemberger and Farley, 2005).

Step 1: Determining system input volume

When the entire system input is metered, the calculation of the annual system input should be a straight forward task. Ideally, the accuracy of the input methods is verified using portable flow measuring device. If discrepancies between meter readings and the temporary measurements are discovered, the problem has to be investigated and if necessary, the recorded quantity has to be adjusted to reflect the real situation. Should there be some unmetered source the annual flow has to be estimated by using any (or a combination) of the following: (i) temporary flow measurements using portable devices,

(ii) reservoir drop test or (iii) analysis of pump curves, pressure and average pumping hours.

Step 2: Determining Authorized Consumption

i) Billed Metered Consumption

The calculation of the annual billed metered consumption goes hand in hand with the detection of possible billing and data handling error, information later on require for the estimation of apparent losses. Consumption of the different consumer categories such as domestic, commercial, industrial) have to be extracted from utility billing system and analysed. Special attention shall be paid to the group of very large consumers.

ii) Billed Unmetered Consumption

Billed unmetered consumption can be obtained from the utilities billing system in order to analyse the accuracy of the estimate; Unmetered domestic customers should be identified and monitored for a certain period, for example by measuring a small area with a number of unmetered customers.

iii) Unbilled Metered Consumption

The volume of unbilled metered consumption has to be established similar to that of billed metered consumption (Farley and Trow, 2003).

iv) Unbilled Unmetered Consumption

Unbilled Unmetered consumption traditionally including water used by the utility operational purpose is very of often seriously over estimated. This might be caused by simplifications (a certain % of total system input) or overestimates on purpose to reduce water losses, components of unbilled unmetered consumption shall be identified and individually estimated, for example

1. Mains flushing: how many times per month? How long? How much water?
2. Firefighting: has there been a big fire? How much water was used?

2.2.3 Quantifying real and apparent losses

Once the volume of (NRW) is known, it is necessary to break, it down in to real and apparent losses, which is always a difficult task.

Step 3: Estimating Apparent Losses

i) Unauthorized Consumption

It is difficult to provide guide line of how to estimate unauthorized consumption. The estimation of such consumption is always a difficult task and should be done in a transparent, component-based way so that the consumptions can later be reviewed. The extent of customer meters inaccuracies namely under or over registration has to be established based on tests of a representative sample of meters. The composition of the sample shall reflect the various brands and age groups of domestic meters based on the results of the accuracy tests, average meter inaccuracy values (as percentage of metered consumption) will be established for different user groups. Data handling errors are sometimes very substantial components of apparent losses (IWA, 2003).

Step 4: Calculating Real Losses

The calculation of real losses in its simplest form is now easy Volume of NRW minus volume of apparent losses-and this figure is useful for the start of the analysis in order to get a felling which magnitude of real losses can be expected. However, it always has to be kept in mind that the water balance might have errors and therefore it is important to verity the real loss figure by one of the following two methodologies (i) component analysis and (ii) Bottom –up real loss assessment.

Step 5: Estimating Real Loss Components

Accurate split of real losses into its components will only be possible with a detailed component analysis However; a first estimate can be made using a few basic estimates.

i) Leakage on transmission and /or distribution mains

Bursts on distribution and especially transmission mains are primarily large events- they are visible reported and normally repaired quickly. By using data from the repair records the number of leaks on mains repaired during the reporting period can be calculated an average flow rate estimated and the total annual volume of leakage form mains calculated as follows: - number of reported bursts x average leak flow rate x average leak during (say 2 days) and then a certain provision for background losses and so far undetected leaks on mains can be added.

ii) Leakage and Overflows at Utility’s Storage Tanks

Leakage and Overflows at Storage Tanks are usually known and can be quantified.

iii) Leakage on Service Connections up to Point of Customer Metering

By deducting mains leakage and storage tank leakage from the total volume of real losses, the approximate quantity of service connection leakage can be calculated, this volume of leakage includes reported and repaired service connection leaks as well as hidden (so far unknown) leaks and background losses from service connections (IWA, 2003),

2.2.3.1 Detailed quantification of real loss components

Step 1: Top-down Water Balance

Although real loss assessment can be done without an annual water balance the total volume of real losses is useful for the start of the analysis in order to get a feeling which magnitude of real losses can be expected.

Step 2: Component Analysis

The key data required for a real loss component analysis of a water distribution system are:

1. Total length of pipe network and number of service connections
2. Average service connection length between curb-stop and customer meter

3. Total number of distribution mains repairs per year (reported and unreported)
4. Total number of service connection repairs per year (reported and unreported)
5. Average system pressure across the entire network
6. Estimates of the time periods for awareness, location and repair duration
7. Estimates of utility storage tank leaks and overflows

Most of this data is readily available in well-organized water utilities: however, the determination of the average pressure across the network is often difficult to estimate.

2.2.3.2 Calculation of average pressure

As the average pressure is a key parameter in any real loss analysis it is certainly worth undertaking some detailed work to obtain a good estimate of the average pressure. Pressures should be calculated as 24 hours averages values (AWWA, 2009).

2.2.3.3 Calculation of background losses

The first of the real loss components calculated are the background losses. Background losses are individual events (small leaks and weeps) that will continue to flow, with flow rates too low to be detected by an active leakage control campaign either unless detected by chance or utile they gradually worsen to the point that they can be detected. Table 2 provides for unavoidable background leakage rates per PSI of pressure at infrastructure correction Factor of 1. (ICF1)

Table 2.1: Unavoidable Background Leakages

Infrastructure Component	Background leakage at ICF =1.0	Units
Mains	9.6	Litres per km per day per metre of pressure
Service connection mains	0.6	Litre per service connection per day per metre of pressure
Service connection –property boundary to customer metre	16.0	Litre per km of service connection per day per metre of pressure

Source: IWA water loss task force, 2003

Unfortunately, the (ICF) is a mostly unknown factor, without carrying our detailed measurements, it is impossible to know the (ICF). In such cases working with the default values of one will means that there is a good chance that the background losses are underestimated and consequently the recoverable losses are overestimated. Using a higher (ICF) (of say 5) might easily lead to an overestimation of the background losses that will cause an underestimation of the true excess loss reduction potential. Thus, it is recommended to work with the (ICF) =1 background leakage value unless better data is available.

2.2.3.4 Calculation of losses from reported and unreported burst

At this Point two definitions have to be introduced

Reported Bursts are those events that are brought to the attention of the water utility conditions, manifests itself at the surface will normally be reported to the water utility.

Unreported Bursts are those that are located by leak detection teams as part of their normal everyday active leakage control duties. After collecting the annual numbers of reported bursts on mains and service connections, flow rates and durations have to be established, unless the utility has investigated average leak flow rates, it is recommended to use the figures from Table 2.2 (Lambert and Lalonde, 2005)

Table 2.2: Average Leakage Rate

Location of burst	Flow rate for reported bursts (l/hr/m pressure	Flow rate for unreported burst (l/hr/m pressure
Mains	240	120
connections	32	32

Source: IWA water loss task force, 2003

The leak duration can be split in three elements time needed for (i) awareness, (ii) location and (iii) repair and estimates will have to be made for each of them.

2.2.3.5 Awareness duration

The awareness duration for reported bursts is generally very short, probably not more than 24 hours. The situation is quite different in respect to unreported bursts, which by definition are detected by active leakage control methods, the awareness time will depend on the active leakage control policy, if for example regular sounding is used and the system is surveyed once a year, the average awareness time will be 183 days.

2.2.3.6 Location duration

The location of a reported leak will in general not take much time since it is visible and a quick check with a ground microphone will be sufficient to verify the leak location, the location duration also depends on the active leakage control policy used (AWWA, 2009).

2.2.3.7 Repair duration

This depends on the utility's repair policy and capacity. Often leaks on mains are repaired within 24 hours but small leaks on service connections within 7 days.

2.2.3.8 Calculation of losses from leaking and overflowing storage tanks

This component has to be dealt with on a case-by -case. Plant operators will normally know if there are problems with overflowing storage tanks, old underground storage tanks may leak, and if this is suspected than level, drop tests could be undertaken. Calculation of excess losses once all the components mentioned above are quantified, the excess losses can be calculated (IWA,.2003).

Step 3 Bottom-up Real Loss Assessment

2.2.3.9 24 Hours zone measurement

Assuming that no district meter areas (DMA) are established, areas of the distribution network have to be selected which can be temporarily isolated and supplied from one to two inflow points only, suitable areas shall be selected in various parts of the distribution system with the objective of obtaining a representative sample of the system. In these area 24-hours inflow measurements will be carried out with portable flow measurement device, these flow measurements shall always be done along with pressure measurements where pressures are recorded at the zone inlet point(s) at the average pressure point and at the critical pressure point, all relevant data on the zone shall be collected, such as (i) length of mains, (ii) number of service connections, (iii) number of household properties and (iv) number and types of non-household properties.

2.2.3.10 Night flow analysis

The minimum night flow (MNF) in urban situations normally occurs during the early morning period usually between around 12 midnight and 04:00 hours, the estimation of the real loss components at minimum night flow is carried out by subtraction an assessed amount of legitimate night consumption for each of the customers connected to the mains in the zone being studied. The result obtained from subtracting these legitimate night uses form the minimum night flow consists predominantly of real losses from the distribution network. The daily level of real losses obtained from the minimum night flow analysis can be determined by applying the (FAVAD) principles (Lambert, 2001) and simulating leakage over the full 24 hours period.

2.3 Causes of Water Losses

Leakage is usually the major component of water loss in developed countries, but this is not always the case in developing or partially developed countries, where illegal connections, meter error, or an accounting error are often more significant (Farley and Trow, 2003). The other components of total water loss are non-physical losses, e.g., meter under registration, illegal connections and illegal and unknown use (WHO, 2001).

2.3.1 Leaks in water distribution systems

Leakage is often a large source of non-revenue water, NRW and is a result of either lack of maintenance or failure to renew ageing systems. Leakage may also be caused for poor management of pressure zones, which result in pipe or pipe-join failure. Although some leakage may go unnoticed for a long time, detection of visible leakage also requires good reporting which also needs a strong public participation. Although leakages after water meter has its own contribution to the overall wastage of water, it is not considered as of the total NRW, as it would be paid for. It is important to distinguish between total water losses (sometimes called NRW and leakage. Total water loss describes the difference between the amount of water produced and the amount which is billed or consumed. Leakage is one of the components of total water lost in a network, and comprises the physical losses from pipes, joints and fittings and also from over flowing service reservoirs (WHO, 2001). The amount of leakage from a reticulation system varies from location to location, due to differences in construction methods, age, and condition.

The condition of the reticulation system is affected by soil movement, corrosive conditions, pipe material, workmanship, age, supply pressure, number of joints and connections, and the occurrence of bursts/cracks result from overburden loading or water hammer, (Mitchell *et al.*, 2000). Leakage reduction as a whole is a complex task which requires coordinated actions in different areas of the water network management such as direct detection and repair of existing leaks, pipe rehabilitation program, pressure control system, etc. and many companies use a mixture of these. Many cities have separated the network into 'leakage districts', and have installed water flow and pressure meters to monitor each district. The registered data are checked and necessary actions taken. Data on bursts and leaks are collected and evaluated to estimate the future need of rehabilitation.

During the last 10-20 years, several cities have started to use computer-based water network records. These databases contain information on network properties, such as pipe material, construction year and diameter and failure information (where, when, failure description,). By simple analyses of these data or by employing more complex statistical methods, information is collected to show differences in failure rate for different pipe properties (Hadzilacos *et al.*, 2002).

Leakage in a network is quantified by a top-down water balance of total water supply against total metered consumption, with allowances for maintenance (i.e., flushing, cleaning), firefighting, metering errors and unauthorised or illegal consumption (Park, 2006); (Rizzo *et al.*, 2004).

Leakage = $TS - MC - [MTAllwnc + FFAllwnc + MEAllwnc + ICAllwnc]$ (2.4)

where: TS = Total supply,

MC = Metered Consumption,

MTAllwnc= allowance for Maintenance,

FFAllwnc = allowance for Fire Fighting,

MEAllwnc= allowance for Metering Errors and

ICAllwnc= allowance for Illegal Consumption.

Leakage in smaller areas can also be quantified by measuring minimum night flows in District Metered Areas (DMAs). After allowances for customer night flow, the balance of the flow is assumed to be due to leakage.

2.3.2 Pressure and leakage

An effective leakage management strategy should take into account the pressure dynamics of a water distribution network. This is because pressure plays a pivotal role in enhancing the magnitude of water leakage. This is because there is a physical relationship between leakage flow rate and pressure. Thus, the pressure exerted by either gravity or by water pumps results in a corresponding change in leakage rate. The frequency of new pipe bursts is also a function of pressure such that the higher or lower the pressure, the higher or lower the leakage. Pressure level and pressure cycling strongly influence burst frequency. Some of the most important ways of managing pressure is by either using pressure reducing valves (manual or automatic) or by using variable speed pump controllers. Under normal circumstances a pressure reducing valve is used to maintain a fixed downstream pressure regardless of the upstream pressure dynamics. The leakage from water distribution systems has been shown to be directly proportional to the square root of the distribution system pressure as indicated by the relationship (Wallingford, 2003).

Evidence shows that the rate of increase of bursts is more than linearly proportional to pressure. Indeed, it has even been suggested that there could be a cubic relationship, i.e. burst frequency proportional to pressure cubed (Farley and Trow, 2003).

Pressure variation in distribution network is caused, among others, by changes of demand of users. The demand usually reaches a peak in the morning when people are at home and preparing their meal and its second peak in the evening. A study conducted in Zimbabwe by (Marunga *et al.*, 2006) also found that with the increase in pressure, there was also an increase in number of bursts. Furthermore, data on changes in break frequency following pressure management in the Bahamas showed that there is a relationship between pressure and burst frequency (Fanner, 2004). Conversely, UKWIR (2005) indicated that there is no evidence of a relationship between pressure and burst frequency. Similarly, Lambert (2001) on investigating data from UK concluded that there is no unique relationship between maximum pressure and new leak frequency, but evidence shows that excess pressures in systems subject to continuous supply, result in higher frequencies and higher repair costs than are necessary. Thornton and Lambert (2005) indicated that the topic was not well studied and the (IWA) Pressure Management Team would seek good quality data of recorded burst frequencies "before" and "after" pressure management in order to improve the current practical methods of analysing and predicting the effect of pressure management on frequency of new bursts, using the provisional relationship that: "burst frequency varies with pressure to the power N2", where N2 is a coefficient relating pressure and burst frequency. N2 values normally ranges from 0.5 to 6.5 as reported by (Thornton and Lambert 2005). Thornton and Lambert (2005) suggested that investigations of the relationship between pressure and burst frequency should be done using the following provisional relationship:

$$BF_1 / BF_0 \;=\; (\mathrm{P1/P0})\mathrm{N2} \quad \mathrm{OR} \quad \mathrm{N2} \;= \mathrm{Ln}\{ {}^{BF_1 / BF_0} / {}_{(\mathrm{P1/P0})} \} \tag{2.5}$$

where: BF_0= burst frequency at initial pressure, P_0

BF_1= burst frequency at the changed pressure, P_1

N2 is burst frequency exponent (coefficient relating pressure and burst frequency).

The determined values were then used to determine pressure management opportunities (Lambert, 2001; Thornton and Lambert, 2005; Fantozzi and Fantozzi, 2008) by computing frequency reduction from possible pressure reduction using the following equation:

$$\Delta\mathrm{BF} = \; 1 - \;\; (\mathrm{P1/P0})\mathrm{N2} \;\; \} * 100\% \;\; = \; \{ 1 - \; (\mathrm{P0} - \Delta\mathrm{P} \;/\mathrm{P0}) * 100\% \tag{2.6}$$

Where: ΔBF is burst frequency, reduction realized upon pressure reduction as a percentage. However, Thornton and Lambert (2007) showed that the N2 approach is inappropriate and recommended that:

1. The N2 approach to analysis should be abandoned as inappropriate
2. Additional "before" and "after" break data should be collected and published

3. An alternative conceptual approach, based on failures being due to a combination of factors, needed to be developed.

As years pass by adverse factors based on age (including corrosion) gradually reduce the pressure at which the pipes will fail (Thornton, 2003). Depending upon local factors such as traffic loading, ground movement and low temperatures at some point in time the maximum operating pressure in the pipes will interact with the adverse factors, and break frequencies will start to increase. This effect can be expected to occur earlier in systems with pressure transients or re-pumping, than in systems supplied by gravity (Thornton and Lambert, 2007). Warren (2005) affirms that the link between burst frequency and pressure is related more to pressure variation, which may also influence the burst shape factor. Overall, there is evidence that there is a relationship between pressure and burst frequency but it may be a complicated relationship.

2.3.3 Ages of pipes and leakage

Although there are no scientifically based criteria for defining the useful life for water mains, there has been a growing concern that many older urban water distributions are deteriorating, massive rehabilitation will be required to replace mains older than some predetermined number of years in age or "useful life" Makaya (2014). Pipe age and material are important factors contributing to the burst probability of pipes that as a result cause lots of water loss. However, as this information is mostly not available especially for aged pipes, it is usually estimated using the history of the urban development. Reports from undertakings collected by the (WRC), and evidence from elsewhere suggest that leakage rates from mains are of the order of 100 to 200 l/hr per km for newer mains and 150 to 3001/hr per km for older mains. Assuming an average of 100 connections per km these figures would represent 1.0 to 3.0 1/hr per connection (Twort *et al.*, 1994). Leakage is frequently the largest component of NRW and includes distribution losses from supply pipes, distribution and trunk mains, services up to the meter, and tanks. The amount of leakage varies from system to system, but there is a general correlation between the age of a system and the amount of NRW. Newer systems may have as little as 5 percent leakages, while older systems may have 40 percent leakage or higher (Walski *et al.*, 2008). Although age is considered as an indicator for predicting the break rate of mains, some studies have shown that it is not the major determinant factor for main water break rates. Poor design, deterioration of pipe material and unanticipated load condition will also result in pipe breakage.

2.3.4 Effects of corrosions on leakage

Corrosion is the problem that is created as water supply pipelines are in continuous contact with soil surrounding it and the water moving through it. The water itself or the surrounding soil may cause problems that will affect the performance and life of the distribution pipes in the system. The majority of the main breaks occur at locations where the pipe wall has been weakened due to corrosion of metal pipes. Corrosion of the external surfaces of cast-iron or steel pipes can, under some conditions, be a significant problem. Therefore, ductile-iron or steel pipelines placed in aggressive soils must be protected by coatings with corrosive resistant materials. The characteristics of the soil in which a pipe is placed affect the rates of corrosion. Recent estimates indicate that the cost of water main breaks in Canada is about $80 million per year. One reason that this cost is so high is that most water mains in Canada are made from either cast or ductile iron. As these pipes age, they are weakened by corrosion, causing an increased number of breaks (IRC, 1996). Designing against corrosion, selection of appropriate materials and usage of protective coating and lining during installation can help for the prevention of corrosion but not limited: Some soils exist in non-corrosive soils too. Soil conditions are responsible for the exterior corrosion of metal structures under or in contact with the ground (Wolday, 2005).

2.3.5 Meter error and water loss

Under registration of customer meters is also one of the causes of water loss. Like the ages of pipes, ages of meters also have an impact to the increase of water loss. Customer meter errors include errors due to accounting procedure and errors due to under or over registration of the meters. Many countries especially developing countries are experienced losses of water due to under registration of meter that many of them put meter replacement policies to alleviate the problem (IWA, 2003).The selection of customer meter types and classes may be limited by water quality considerations, as well as technical and economic considerations, economic replacement policies for residential. Meters based on selective testing programmes in the National Reports generally indicate changeover periods between 5 and 10 years.

Where customers are served by way of roof tanks, the probability of customer meter under-registration is increased, because of the tendency for a greater part of the consumption to pass through the meter at rates less than the Q minimum specified for the meter (Liemberger *et al.,* 2002). The cities of Africa appear to use meters for 78 % of domestic consumption and the yearly meter replacement is about 8.8 %. Considering that meters typically under read as they age, it is likely that considerable proportion of unaccounted for water is experienced by metering errors (WHO, 2000). Domestic water meters tend to under register for two reasons, i) malfunctioning due to deterioration with use, and ii) inability to measure low flows accurately. Much larger under registration can occur when maintenance of meters is poor Twort *et al.* (1994).

2.4 Consequences of Water Loss and Leakage

The primary consequences of leaks in a distribution system are financial and operational challenges. Reduction in water losses enables water utilities to use existing facilities efficiently, alleviate shortage of water supply, improving the supply capacity to consumers and the reduction of operational expenditures that are related to power and chemical costs. Reduction of water losses extends the service life of existing water supply components that as a result to meet the present as well as the future needs of residents without construction of any new water facilities. Beside to low revenue generation as a result of under-recording of faulty meters, or totally uncharged due to illegal connections and unregistered consumption leakage also greatly contributes to loss of revenue.

The operation and maintenance costs including price of energy, chemicals and other items that are constantly rising will also be aggravated by the increase of water loss due to leakage. Beside affecting production and management costs, leaks have great consequence on the quality of services. The water that escapes from leaks may also cause a damage of structures such as sinking of roads and other properties. When the leak becomes more serious or a pipe bursts, service may be interrupted totally that many people will be severely affected.

2.5 Leakage Monitoring and Control

The losses of water are inevitable in the process of supplying thousands of customers spread over a large area started from reservoirs at strategic locations, through a complex network to the individual customer. Leakage monitoring and control in pipe reticulation systems is critical in ensuring the efficient performance of the system. Pipe systems are commonly used for distributing water to areas of consumption. If pipes are worn out, large volumes of treated water may be lost through leakage as a result of high pressures of flow. Leakage control is possibly one of the most difficult tasks for water engineers. Even in developed countries, about 15-20% of the distributed water is lost through pipe leakage. It is therefore important to ensure that leakage monitoring and control is given the attention it deserves by all water supply authorities and consumers (Mulwafu *et al.*, 2003). Water leak detection is a systematic method of locating visible and non-visible leaks in a distribution system through visual inspection; pipe locators and leak detection equipment (proactive leak detection); and pressure control, etc. Depending on the type, leakage could be identified from the simplest of using visualization till using sophisticated equipment as discussed below.

When a district metered area (DMA) is incepted its respective NRW values should be calculated, the Net Night Flow (NNF) and apparent losses, and identify the main areas of concern USAID and WBI (2010). Once the DMA leakage is found to be high, respective NRW reduction activities should be implemented. It can be shown that the NRW level within a DMA does not remain the same, but rather as the infrastructure variables like age of the pipeline, the wear and tear of the components of the network and system's pressure dynamics change, so does NRW (Farley, 2003). Therefore, it is the prerogative of the water utility to ensure that the major components of NRW, physical losses are monitored accordingly. The calculation for NRW within a DMA is defined as follows:

DMA NRW = Total (DMA) Inflow - Total DMA Consumption.

The level of metering of a (DMA) generally affects the water consumption. The higher the water meter density, the higher the consumption. Thus, if all consumers in a DMA are metered, the total consumption is the sum total of all individual meters while for systems partially metered, (DMA) consumption is approximated using per capita consumption. In such case further information about water demands and average per capita values are needed (Farley *et al.*, 2008)

Assessment of pressure and flows within a water supply area can be done by employing data loggers designed to record the pressure and flows, a method popularly known as night flow analysis (McKenzie, 1999). The minimum night flow (MNF) methodology is the best practice analysis and monitoring strategy for water leakage within a District Metered Area (Hunaidi, 2010). The analysis is done when the customer demand is at its minimum and therefore the leakage component is at its largest percentage of the flow. The MNF method puts emphasis on problematic areas with a high percentage of NRW in real time especially if night consumption is expected to be fairly small. Normally, it presents the flow starting from 12 midnight to 04:00 hours where most of the consumers are inactive. Unfortunately, not all water utilities are privy to these methodologies for various reasons. Some of the reasons include poorly laid out distribution networks, non-availability of pressure and flow logging equipment, and lack of skilled personnel to do the analysis.

The South Africa Night Flow (SANFLOW) analysis model is based directly on the BABE (burst and background estimate) and Fixed Area Variable Area Discharges (FAVAD) principles and is written in DELPHI computer language for the Windows operating system (McKenzie, 1999). It includes the ability to undertake sensitivity analyses based on basic risk management principles in order to provide a likely distribution of the number of bursts in a zone (or district). By using the sensitivity analysis feature of the model, potential problems can be addressed. The methodology used in (SANFLOW) is a very empirical method based on a large number of test results from the United Kingdom and elsewhere in the world (McKenzie, 1999). Despite the empirical approach, the methodology has been used with great success in many parts of the world including Europe, the Middle East, Malaysia, South America, Africa and the United States of America. Apart from South Africa, Chiipanthenga (2008) and Chipwaila (2009) used the model in Malawi successfully. From such registered successes, water losses in developing countries can be reduced to economic levels and water utilities in these countries can enhance their service delivery systems (Makaya and Hensel, 2014). Thus, developing countries should also embrace the relevant technological tools that would be very helpful in planning water loss management as well as operation and maintenance programmes.

2.5.1 Identifying leaks through visual inspection

In this method only those leaks that become self-evident are located and repaired. A leak may be self-evident because water shows on the surface or may become so upon investigation following consumer complaints such as poor pressure or noise in the plumbing system (Walling, 2003). Bursts of large mains are often visible and are not considered as major causes of high-water losses as these incidents are quickly spotted and repaired or isolated. This method is widely applied and requires regular inspection by the respective authority and it does not need special professional skills.

2.5.2 Identifying leak using detection equipment

Most of the water is lost through numerous small holes, which are very difficult to locate, as the pipes are laid underground that usually need special equipment to locate the leak and repair. This method involves teams of inspectors seeking to locate leaks by systematic direct sounding on all stopcocks, hydrants and valves through the distribution system and listening for the characteristic noise of leaking water. As water under pressure exits a crack or a small hole, the pipe wall and the surrounding soil emit sound waves in the audible range. Water impacting the soil and circulating in a cavity creates lower frequency waves that have limited transmission through the ground. Through the use of surface microphones, leaks can be located with greater precision. The leak noise detected will depend upon the position at which a sounding is made (Wolday, 2005).

2.5.3 Location of large leaks by pressure control

A large leak in a small network can be located by measuring the pressure during the time of minimum water supply especially during the night. This can be done by shutting of the valves in successive sections of the distribution starting from the supply. Pressure control does not directly involve leakage detection, but sudden drops in pressure may indicate to a possible leak. In general, reduction in pressure leads to reduced rate of escape through each leak and may also affect the number of leaks occurring. Pressure reduction is relatively cheap and can be quickly affected, but lower pressure may also increase the leak population by making them less detectable. Pressure reduction can be achieved in a number of ways such as reducing pumping heads, installing break pressure tanks and using pressure reducing valves. The control of pressure surges and cycling is likely to reduce the numbers of bursts and leaks that occur, especially in plastic pipes.

2.6 Computational Approaches to Leak Detection

This section presents features of EPANET as it relates to water distribution analysis.

2.6.1 Network hydraulic modelling in leakage assessment

Most software widely used water distribution network simulation requires that sub-components for distribution storage and piping be inputted with the necessary information. Example of such software is EPANET which was developed by Environmental Protection Agency of USA. Pipes, represented as links, require the size, length, and roughness (i.e., Hazen-Williams C- factor) of a pipe be entered. Additionally, valves must have the correct size and operating conditions inputted. Further, tanks in EPANET need to be entered with the correct dimensions (i.e., diameter) and operating conditions such as minimum water level, maximum water level, and starting water level. These conditions allow tanks to function as floating tanks, because during the course of a simulation, a tank may fill up or supply the distribution system depending upon current demands. Pumping stations are rather complicated containing both sub-components and sub-sub-components. In EPANET, pumps are simulated mainly using a pump curve that relates the pressure head to flow. These curves allow pumps to function within the manufacturer's specifications.

Pumps are also controlled by other operating conditions such as tank levels and nodal pressures through the use of controls and time patterns. To determine the operating point on a pump curve, a relationship between the system and pump curves must be made. The system head curve is a function of the pipe network in which the pump is located and represents the resistance that the pump must overcome. The following equation is used to determine the system head curve (Rossman, 2000);

$$h_{system} = h_{stat} + h_f + h_{ML} = \Delta z + h_{req} + h_f + h_{ML} \tag{2.7}$$

Where:

h_{stat} = Static head (L)

h_f = Friction head loss (L)

h_{ML} = Minor head loss (L)

Δz = Change in elevation (L)

h_{req} = Required head (L)

The relationship that results between the system head curve and pump curve (Figure 1.2 below) provides insight into the operation of the pump. As seen in Figure 2.2, the intersection of the two curves is the point at which the pump operates

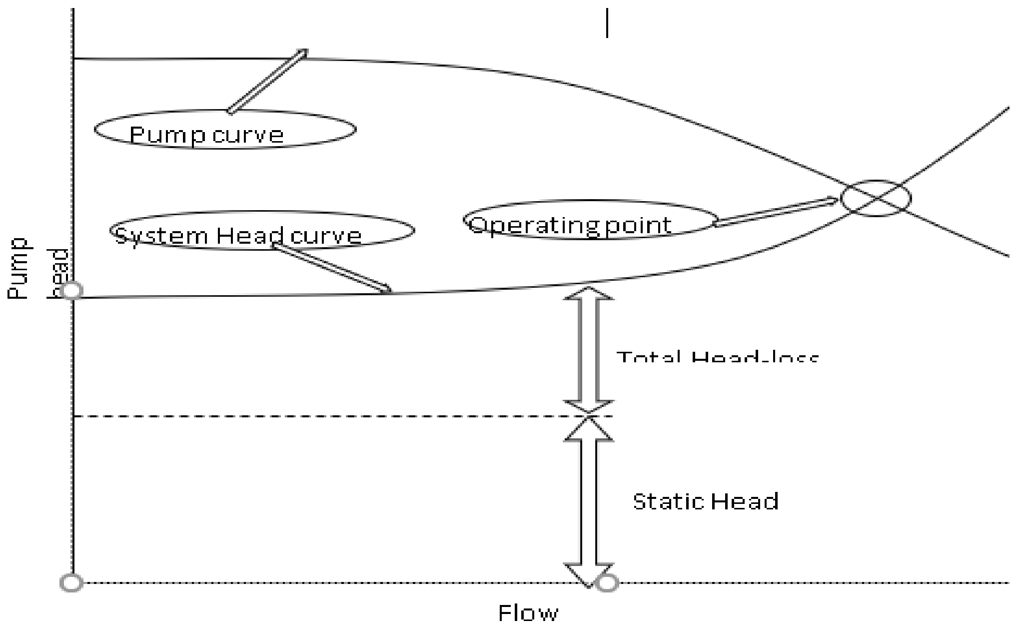


Figure 2.2: Relationship between pump and system curve
(Adapted from Boulus *et al.*, 2008)

2.6.2 Importance of EPANET

EPANET is a computer program that performs extended period simulation of hydraulic and water quality behaviour within pressurized pipe networks. A network consists of pipes, nodes (pipe junctions), pumps, valves and storage tanks or reservoirs. EPANET tracks the flow of water in each pipe, the pressure at each node, the height of water in each tank, and the concentration of chemical species throughout the network during a simulation period comprised of multiple time steps. In addition to chemical species, water age and source tracing can also be simulated.

EPANET is designed to be a research tool for improving our understanding of the movement and fate of drinking water constituents within distribution systems. It can be used for many different kinds of applications in distribution system analysis. Sampling program design, hydraulic model calibration, chlorine residual analysis, and consumer exposure assessment are some examples.

Running under Window EPANET provide an integrated environment for editing network input data, running hydraulic and water quality simulations, and viewing the result in a variety of formats. These include colour-coded network maps, data tables, time series graph and contour plots.

EPANET contains a state-of-the-art hydraulic analysis engine that include the following capabilities:

1. Places no limit on the sizes of network that can be analysed
2. Computes friction head loss using the Hazen-Williams, Darcy Weisbach, or Chezy-Manning formulas
3. Includes minor head loss for bends, fittings
4. Models constant or variable speed pumps
5. Computes pumping energy and cost
6. Models various types of valves including shutoff, check, pressure regulating, and flow control valves
7. Allow storage tank to have any shape (i.e., diameter can vary with height)
8. Consider multiple demand categories at nodes, each with its own pattern of time variation
9. Model pressure-dependent flow issuing from emitters sprinkler head)
10. Apply system operation on both simple tank level or timer controls and on complex rule-based controls. One of the challenges of EPANET is it takes a long time to learn.

2.6.3 Head-loss equations

Major losses occur due to friction within a pipe. Minor head losses are caused by the added turbulence that occurs at bends and fittings (Rossman, 2000). The three most common equations are the Manning, Hazen-Williams and Darcy-Weisbach equations. The Manning equations is more typically used for open channel steady and unsteady flow and is dependent on the pipe length and diameter, flow and the roughness coefficient (Manning roughness). The following is the Manning equation (Walski *et al.*, 2008).

$$h_L = {C_f L (nQ)^2}\Big/{D^{5.33}} \tag{2.8}$$

Where n = Manning roughness coefficient

C_f = Unit conversion factor (English = 4.66, SI = 10.29)

L = Pipe length (L)

D = Pipe diameter (L)

Q = Pipe Flow (L^3/T)

The Hazen-Williams equation has been used mostly in North America and Africa and is distinctive in the use of a C-factor. The C-factor is used to describe the carrying capacity of a pipe. High C-factor represents smooth pipes and low C-factors represent rougher pipes. The following is the Hazen-Williams equation (Walski *et al.*, 2008).

$$h_L = \frac{C_f L}{C^{1.852} D^{1.852}} Q^{1.852} \tag{2.9}$$

where C = Hazen-Williams C-factor

The Darcy-Weisbach equation was developed using dimensional analysis. This expression uses many of the same variables as the Hazen-Williams equation, but rather than using a C-factor it uses a friction factor, f. The following is the Darcy-Weisbach equation (Walski *et al.*, 2008).

$$h_L = \frac{8fLQ^2}{gD^5\pi^2} \tag{2.10}$$

where

f = Darcy-Weisbach friction factor

g = Gravitational acceleration constant (L/T^3)

Several different methods have been developed for estimating the friction factor, f. Two of the main methods are the Colebrook-White and Swamee-Jain equations. The Colebrook-White equation is one of the earliest approximation methods that relate the friction factor to the Reynolds number and relative roughness. The following is the Colebrook-White equation (Walski *et al.*, 2008).

$$\frac{1}{\sqrt{f}} - \; -0.86 \ln\left(\frac{\varepsilon}{3.7D} + \frac{2.51}{Re\sqrt{f}}\right) \qquad (2.11)$$

where:

ε = Equivalent roughness

Re = Reynolds number

The main issue with this equation is that the friction factor is found on both sides of the expression. This requires one to solve the expression iteratively to determine which value of the friction factor satisfies the equation. This resulted in the development of the Moody diagram which is a graphical solution for the friction factor. The Swamee-Jain equation is considered to be much easier to solve than the iterative Colebrook-White equation. The following is the Swamee-Jain expression (Walski *et al.*, 2008).

$$f = \frac{1.325}{\left[\ln\left(\frac{\varepsilon}{3.7D} + \frac{2.51}{Re\sqrt{f}}\right)\right]^2} \qquad (2.12)$$

The relative simplicity and accuracy of the Swamnee-Jain equation has influenced water distribution system model developers to use this equation to solve for the friction factor.

To better understand certain advantages and disadvantages between the Darcy-Weisbach and Hazen-Williams solutions a study was conducted that compared the results of a flow model using these two head-loss equations. This study compared the Colebrook-White and Hazen-Williams flow models in a real-time water network simulation. The Colebrook-White equation was the method used to determine the friction factor for the Darcy-Weisbach equation. The Hazen-Williams method is more advantageous to the

Colebrook-White method due to its simplicity. However, problems arise due to the approximation solution formed by the Hazen-Williams equation, mainly because of the wide range of flows that exist in a real-time water distribution network. The Colebrook-White equation has been widely accepted as more suitable for determining an accurate solution when a wide flow range is present. The Hazen-Williams approach has a time saving advantage over the Colebrook-White method in that the pipe resistance (C-factor) is not a function of flow. Since the Hazen-Williams equation does not account for water temperature, it is not very suitable for carrying water conditions. The Colebrook-White equation on the other hand is explicitly dependent on the kinematic viscosity of water which is a function of temperature. This attribute makes the Colebrook equation suitable for a water network simulation that has varying water conditions.

With the advent of the information and communication technology, computation methodologies have been developed and these methodologies are taking their toll in many water utilities. Among the most widely used water leakage assessment tools is Network Hydraulic Modelling (NHM) is finding wide application world over. The method uses computational strength of computers for forecasting and investigating the operational functionalities of (WDS). EPANET 2 is one other extensively used network hydraulic modelling (NHM) software found on public domain (Rossman, 2000). Its hydraulic solver is based on an expandable open-source code using the gradient method. In leakage management the NHM can be can be used for pressure management, network zoning and decision making about pipeline replacement (Wu *et al.*, 2011). Germanopoulous (1985) applied empirical functions to correlate water supply with network pressures. In such an application functions in the mathematical formulation were used for network analysis. As a result, the pressure-consumption relationship for a given node was expressed as:

$$C_{i=}C_i^k a_i \, e - ^{b_{iP}}{}_{i/P_i^k} \qquad (2.13)$$

where: Pi = pressure at node i.

Ci = the consumer outflow at node.

i; = the nominal consumer demand.

ai,, = constants for the particular node.

The network model includes leakage using

$$Vij = c(LijPijav)N1 \tag{2.14}$$

where: Vij= leakage flow rate from the pipe connecting nodes i0

j; cl= a constant depending on the network;

Lij= pipe length

$P_{ij}{}^{av}$ = average pressure along the pipe

N1 = the pressure exponent

An extension of the method was done Vela *et al.* (1991) factoring pipe size and condition parameters as shown in Equation 2.9

$$V_{ij} = Ci\left(L_{ij}D_{ij}^{d}\, e^{a\tau}P_{ij}^{av}\right)^{N1} \tag{2.15}$$

where "D and τ are pipe diameter and age respectively; d is 1 for (D < 125 mm) and is -1 for (D >125 mm); and a is a leakage shape parameter". The only draw-back about this methodology is the required data.

The field measurements required to determine parametric values of ai, k for every node is the major disadvantage of the method. As a result, many utilities in developing countries cannot afford the cost of experimental procedures. Another disadvantage of the method is that leakage flow is assumed to be uniformly distributed along the pipeline. This, in reality is not the case because of the differences in types of pipeline materials and positioning of joints and fittings. Rossman (2000) proposed another approach where leakage is assumed to behave as an orifice flow. According to EPANET 2, the respective pressure-dependent outflow relationships shown in Equation 2.14, allows for leakage modelling in emitter nodes.

$$Q_{i,i}(t) = K_i[P_i(t)]^{N1} \quad (2.16)$$

where $Q_{i,}(t)$ is the leakage aggregated at node i at time t; $P_i(t)$ is the pressure at node i at time t and K_i is the emitter coefficient for the node i, and a positive K_i is an indicator of leakage demand at node i. Whereas the aforementioned methods for leakage assessment are of great importance, their main drawback is that they cannot pinpoint the actual location of the leakage.

According to Ayanshola and Sule (2006), the minimum residual pressure for pipe network systems pressure varies from one water agency to another and among countries. Bhardwaj (2001) recommended a minimum pressure head of 25m and maximum of 70m by Arunkumar and Mariappan (2011). The Central Public Health and Environmental Engineering, New Delhi, India (1999) stipulated a minimum pressure of 12m at destination junctions. About 35m and 140m were recommended by Washington State University Uniform Design Standard (1998). However, due to lack of local standards, the American Water Works Association (1956) recommendation of a minimum of 15m and a maximum of 70m was adopted. Any pressure less than 15m is deemed insufficient and might infer total water loss at node. In the same vein, where the pressure is more than 70m, water hammer occurs where the water mains are susceptible to breaks and damages (Bwire *et al.*, 2015).

Also, according to the United Kingdom's Office of Water Services (OFWAT, 2010), a flow rate of at least 0.15 liters per second in WDS is acceptable. At a very low velocity suspended solids start to settle and accumulate at the lower ends or areas reducing the actual diameter of the pipes. Regions close to 0.5 m/s indicates pipes are self-cleansing. Velocities above 3m/s indicates pipes are too small and create a risk of damage by water hammer. Higher values of unit head loss above 10m/km indicates in inefficiency in that particular pipe. However, it is desirable to have high values of frictions in some cases to dissipate pressure in low points in the network. This allows savings by maintaining the pipes (pipes considered small) and prevents excess pressure provided there is no anticipation of future extension.

2.7 Water Loss Analyses

The quantity of leak in circular holes for the transmission, distribution, and connection pipes lines to be calculated as a function of cross-sectional area of pipe and pressure head. Leakage management practitioners recognize that all water supply systems leak to some extent and it is impossible to totally eliminate real losses from a large water distribution system (Thornton, 2003). The lowest technically achievable annual volume of real losses for well-maintained and well-managed systems is known as unavoidable annual real losses (UARL), (Radivojević *et al*., 2007). (UARL) is defined (Cakmakcı *et al*., 2007) as that portion of underground system leakage that is considered not economical to locate and repair or too small to detect using current technology. System-specific values of UARL can be assessed using a formula developed by the (IWA) Water Losses Task Force (Lambert *et al.,* 1999). Data required for this assessment are the number of service connections (Nc), the length of mains (Lm in km) and the length of private pipes (Lp in km) between the streets, property boundary and customer meters, and the average operating pressure (P in metres). According to Lambert and Lalonde (2005) the general equation for UARL calculations is:

$$UARL = (18\,x\,Lm + 0.8\,x\,Nc + 25\,x\,Lp)\,x\,P \tag{2.17}$$

Where:

UARL = Unavoidable Annual Real Losses (L/d)

Lm = Length of mains (km)

Nc = Number of service connections (main to meter)

Lp = Length of unmetered underground pipe from street edge to customer meters (km)

P = Average operating pressure at average zone point in meters

The equation is based on component analysis of real losses for well-managed water distribution systems with good infrastructure and has proven to be robust in diverse international situations Lambert and McKenzie, (2002). It is the most reliable predictor yet of "how low you could go" with real losses for systems with more than 5000 service connections, connection density (Nc/Lm) more than 20 per km and average pressure more than 25 metres (Fanner, 2004; Seago *et al.,* 2004). From these figures, it implies that the equation cannot be used for (DMAs) with less than the stipulated values. Lambert and Lalonde (2005) recommended that for systems operating not at the standard pressure of 50m, there is need to revise the (UARL) by the following equation:

$$UALr \ = \ UARL50 \, x \ (P50)1 \qquad (2.18)$$

The current (UARL) then becomes the revised UARL, while UARL50 is UARL at standard pressure of 50m, P is operating pressure, and N1 is leakage exponent (coefficient relating pressure and leakage). The current annual real loss (CARL) comprises of physical water losses from the pressurised systems through to customer water meter, and is normally calculated as the total water lost less the apparent losses (Seago *et al.*, 2004).

2.8 Leakage Management and Control

2.8.1 Passive leakage control

Passive leakage control is reacting to reported bursts or a drop in pressure, usually reported by customers or noted by the utility’s own staff while carrying out duties other than leak detection (Farley, 2003). The method can be justified in areas with plentiful or low cost supplies. McKenzie (2002) indicated that this type of leakage control is often practised in less developed supply systems where the occurrence of underground leakage is less well understood. Except in exceptional circumstances, leakage will continue to rise under passive control (McKenzie, 2002).

2.8.2 Active leakage control

Mathis *et al.* (2008) defined Active Leakage Control (ALC) as any water utility program that proactively seeks to discover leaks which have not been reported by customers or other means. The most typical methods of active leakage control are routine leak detection surveys and the use of minimum night flow measurement in DMA or pressure zones (Mathis *et al.*, 2008). The usage of the (DMA) approach to leakage detection and localisation has become an international best practice. Sturm and Thornton (2007) indicated that the most appropriate leakage control policy will mainly be dictated by the characteristics of the network and local conditions, which may include financial constraints, equipment and other resources. In many developing countries, the method of leakage control is usually passive or low activity, mending only visible leaks (McKenzie and Bhagwan, 2004).

A simple methodology was presented by (Gianfredi *et al.*, 2016) by which the analyses of inflow data records were collected in several water distribution networks. Leakages were assessed based on the seasonal fluctuation of water consumptions. The methodology was tested on two synthetic case studies based on the Apulian region WDN in Italy, where hydraulic status was simulated by an advanced WDN model that included a realistic pressure-dependent background leakage model. The analyses of case studies verified the effectiveness of the methodology under fully controlled WDN configurations (e.g., neglecting measurement inaccuracies that might happen in real WDN and/or possible alterations of asset conditions over the analysed period). The resulting estimates of leakages proved to be accurate under the analysed condition (Gianfredi *et al.*, 2016)

The paper provided the understanding of operating principles of currently available pipeline leak detection technologies. The distribution pipelines in urban water supply need to be monitored for contaminants such as microbial growth, internal corrosion of the pipe's material and other deposits. In addition to the loss of water resources, the contaminant can be infiltrated into the piping system. These contaminants affect not only the quality of the water but also the smoothness of the water pipe flow due to the pressure loss and additional frictions. (Turki *et al.*, 2020)

2.9 Artificial Neural Network

2.9.1 Artificial neural network as an analytical tool

Artificial Neural Networks (ANN) comprise of a network of neurons and take the cue from their biological counterparts. (ANNs) have found wide application in modelling water resources management problems including leakage detection, water distribution network optimisation, water pipeline replacement and rehabilitation, water demand forecasting, and pressure monitoring.

Neural networks are made up of a series of layers with each layer comprising at least one neuron (Shamseldin, 1997). While intermediate layers (hidden layers) perform the data, processing functions of the network, the first and last layers input and output variables respectively. Within the hidden layers, weights to the neurons are adjusted by training the network in accordance with the stipulated learning rule (Zealand *et al*., 1999). The neuron transfer function plays the role of transforming the input to output for each neuron. The log-sigmoid transfer function is commonly used for hidden layer neurons; especially with the back-propagation algorithm (The Mathworks, 2002). Back propagation algorithms are based on multi-layered feed forward topology with supervised learning.

An optimisation algorithm is used to select the control input that optimises future performance. Figure 2.3 depicts a typical example of one hidden layer feed-forward neural network architecture.

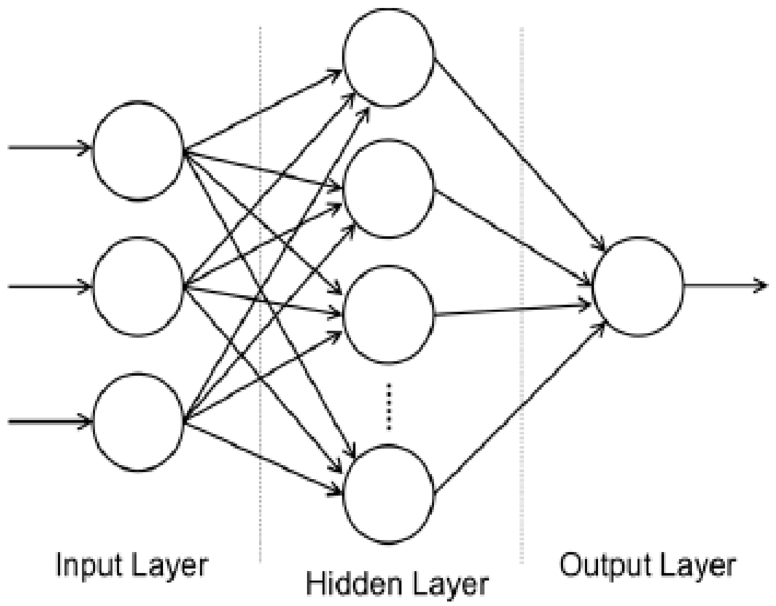


Figure 2.3: Schematic Diagram of Multilayer Feed-Forward Neural Network (Haykin, 1994)

An artificial neural network (ANN) is a model used for predicting dependent variables through statistical learning algorithms when sufficient data on independent variables are available to describe dependent variables. Major (ANN) studies applied to water distribution systems in recent years are as follows. A procedure to devise a general operating policy toward reservoir operation from a dynamic programming using neural network (DPN) was suggested (Raman and Chandramouli, 1996). Relatively new technique of using (ANNs) researched for forecasting short-term water demand (Jain *et al.*, 2001). (ANNs) in water quality modelling, as well as for the process and control of treating drinking water used in water distribution systems (Baxter *et al.*, 2001). Research on the application of (ANNs) for analysis of data from sensors measuring hydraulic parameters is presented (Mounce and Machell, 2006). Additionally, the efficiency of computational intelligence techniques was compared in water demand forecasting (Msiza *et al.*, 2008). Recent research about (ANN) used it as a means of estimating the temporal variation of analytic factors such as real-time water quality, operation of reservoir and short-term demand forecasting.

The application of an (ANN) to water distribution systems for estimating NRW and parameter analysis, however, proved insufficient. An (ANN) is a massively parallel distributed processor with a natural propensity for storing experiential knowledge and making it available for use. It resembles the human brain in two respects: knowledge is acquired by the network through a learning process and inter-neuron connection strengths, known as synaptic weights, are used to store the knowledge (Haykin, 1994). The ANN procedure used is a feed-forward network type with input, hidden and output layers, as shown in Figure 1. Neurons in the input layer simply act as a buffer. Neurons in various layers are interconnected through weights. Neurons in the hidden and output layers are called the activation function, and the activation function used here is a sigmoidal activation function. The input for each neuron j in the hidden layer is the sum of the weighted input signal xi. Expressed as $w_{ji}x_i$ = net_j, in which w_{ji} is the interconnecting weight between neuron j in the hidden layer and neuron i in the input layer.) The output yj from the neuron given by the neuron output in the output layer is computed similarly.

$$yi = f\,(\,\text{å}wjixi) = 1/(1 + e) - netj \qquad (2.19)$$

2.9.2 Prediction ability of ANN

Ostadi and Azimi (2015) in predicting the price of steel using ARIMA, found that ANN are good at tackling the problem of over-fitting, neural network prediction error was less than the usual method of (ARIMA), which shows the high performance and power of this method in predicting. Zhang *et al.*, (1998) indicate that, as opposed to the traditional model-based methods, (ANNs) are data-driven self-adaptive methods in that there are few a priori assumptions about the models for problems under study. ANNs learn from examples and capture subtle functional relationships among the data even if the underlying relationships are unknown or hard to describe. Artificial neural networks, which are nonlinear data-driven approaches as opposed to model-based nonlinear methods, are capable of performing nonlinear modelling without a priori knowledge about the relationships and generally are a flexible modelling tool for forecasting.

Hamideh *et al*. (2021) proposed a new method to locate a leakage in WDNs using feedforward artificial neural networks ANNs. For this purpose, two ANNs training cases were considered. For case 1, the ANNs were trained by average daily water demand, including small to large hypothetical leakages. In case 2, the ANNs were trained by hourly water demand and variable hourly nodal leakages over 24 hours. The training parameters were determined by EPANET 2.0 hydraulic simulation software using MATLAB programming language. In both cases, first, ANNs were trained using flow rates of total pipes number. Then, sensitivity analysis was performed by hybrid ANNs for the flow rates of pipes number less than the number of the total pipes. The results of proposed hybrid (ANNs) indicate that if at least the flow rates of 10 % of the total pipes were known (using flow meters), then the leakage locations in both cases could be determined. Despite the complexity of case 2, because of the variations of demand and leakage over the 24-hour period, the proposed method could detect the leakage location with high accuracy. The operational parameters: Number of leaks, demand energy ratio, mean pipe diameter, pipe length, demand junction, water supply quantity and deteriorated pipe ration were used as parameters that affect water distribution system for the estimation of NRW using Artificial Neural Network and Multiple Regression Analysis. (Dangwoo and Gyewoon, 2017). Based on the results of the previous study, calculation of the (NRW) ratio is recommended by using MRA, which is obtained from the physical and operational parameters in water distribution networks. This study tried to use an (ANN) for estimating the NRW ratio, then compared the results of (MRA) and the (ANN). An accuracy assessment showed that the (ANN) model had higher prediction accuracy than that of (MRA). A methodology has been developed for estimating the NRW ratio using an ANN with the main parameters of water distribution systems. When an ANN was used, the accuracy of NRW ratio estimation was higher than under the previous method of (MRA). So, when it is difficult to measure the NRW ratio and use MRA in a DMA, the AN) model is recommended for estimating the NRW ratio using the main parameters of water distribution systems. Detection and localization of leaks for Smart cities was conducted using Supervisory Control and Data Acquisition (SCADA) and Geographical Information System, GIS and hydraulic model of the water supply network, an algorithm of leak detection and location based on the neural networks' multi-layer perception. The algorithm has been tested on the hydraulic models of several municipal water supply networks Izabela and Jan (2019).

The verification method for the results of hydraulic calculations, with the use of process diagnostics and artificial neural networks, is presented in the paper Dawidowicz (2017). The method for estimating pressure levels and the pattern of pressure zones, using artificial neural networks, is described in the article Dawidowicz (2017) and using the induction method of the decision tree at work Dawidowicz (2012). The problem of the assessment of pressure loss, is discussed in papers (Biedugnis and Czapczuk, 2018), in which different methods of artificial intelligence have been used, including expert systems and the method of k-nearest neighbours. In this work, artificial neural networks of the perceptron type have been used for the above purpose.

In the assessment of pressure loses in the water pipe of distribution system, solution is proposed to indicate the pipelines where it would be possible, or recommended, to adjust the diameter, in order to ensure adequate linear pressure losses and therefore more favourable operating conditions, from the point of view of the network pressure level, using artificial neural network Jacek *et al.* (2018)

2.9.3 Applications of ANN

Neural networks operating on quasi-static pressure and flow readings have been used for leak detection in pipe systems. Makaya and Hensel (2015) developed a methodology by using ANN to model leakages in the water distribution network of the City of Harare. Caputo and Pelagagge (2003) have described an approach to detecting spills and leakages from pipeline networks using a multilayer perceptron back-propagation (ANN). In order to determine the location and size of leaks in the pipe network, a two-level architecture composed of a main (ANN) at the first level and several branch specific second level ANNs were used. The branch in which the leakage occurs is estimated by the main ANN while a specific second level (ANN) is activated to estimate the magnitude and location of the leakage in the selected branch.

A similar approach utilising pressure reading only was described by Shinozuka *et al.*, (2005). The methodology described, identified the location and severity of damage in a water delivery system by monitoring water pressures on-line at some selected positions in the system. Another application of (ANNs) operating on steady state process parameters for leak detection in pipe systems was delivered by Belsito *et al.* (1998), describing an approach to leak detection in liquefied gas pipelines.

A neural network for leak detection operating on sound signals emanating from a pipe network was used by (Zhang, 2004). Their work described a method for detecting gas leaks in pneumatic pipe systems. A system using fuzzy logic acting on flows in a pipe system for leak detection was described by De Silva *et al.* (2011). Their system was used for the detection of leaks in petroleum pipelines. The authors reported an accuracy level greater than 90 % in leak detection. Feng and Zhang (2006) described an approach to pipeline leak detection using a Discrete Incremental Clustering fuzzy (ANN).

Another fuzzy (ANN) system for fault detection in water supply systems was described by Izquierdo *et al.* (2007). Their method was based on a mathematical model of the system and the application of a fuzzy neural network was found that this system had good classification accuracy for large leaks.

Three algorithms, differential evolution (DE), artificial bee colony (ABC) and ant colony optimization (ACO), were used to determine the optimal one for forecasting downstream river flow. A hybrid neural network (HNN) model, which incorporated fuzzy pattern-recognition and a continuity equation in to the artificial neural net-work, was proposed to forecast downstream river flow based on upstream river flows and areal precipitation. The three algorithms presented stability and reliability with respect to their control parameters on the whole (Chen *et al.*, 2015)

The process of creating a set of training data and searching for the appropriate structure of an artificial neural network is complicated and time-consuming. Training artificial neural networks should be carried out repeatedly, in order to avoid the local minimum of the error function. The artificial neural network was developed for computer programmes, in order to calculate hydraulic water distribution systems, in which it acted as an additional module, in the assessment of the results obtained. After completing the calculations, additional DH1-DH5 classes was assigned to each calculation section. The proposed solution was to indicate the pipelines where it would be possible, or recommended, to adjust the diameter, in order to ensure adequate linear pressure losses and therefore more favourable operating conditions, from the point of view of the network pressure level. The neural network obtained was highly accurate at classifying. (Jacek *et al.*, 2018).

This study conducted in the Konyaalti Water Distribution Network in Antalya, Turkey, (Karadirek *et al.*, 2012) The study area was divided into 18 district metered areas (DMAs) for better management of water losses. Water levels in reservoirs, flow rates, and water pressures were monitored on-line by the SCADA data system. A hydraulic model was calibrated and verified for each DMA using data provided by SCADA. The model results revealed that a number of (DMAs) exhibited high pressures, greater than 3.5 bars, and high minimum night flow throughout the year. Also, the Infrastructure Leakage Index for the study area was greater than 20, indicating high water losses. As a result of these findings, a pressure reducing valve (PRV) was installed at (DMA) No. 2 as an example and set at 3.0 bars resulting in considerable reduction in water losses. The optimum pressure level for setting the (PRV) was chosen using the hydraulic model. The same model was used to predict water savings due to pressure reduction. The predicted water savings were verified using long periods of flow rates and water pressure profiles. The predicted and measured water savings showed good agreement. The study concluded that hydraulic modelling is essential for applying appropriate pressure management strategies (Karadirek *et al.*, 2012).

Soldevila *et al.* (2019) have presented leak localization method using Kriging Interpolation in which the node with the highest difference in current and non-leakn pressure values were identified as the leak nodes. The pressure of limited pressure sensor in the water distribution network has been addressed

2.9.4 Advantages of ANN

Maind and Wankar (2014) laid out the following advantages of ANN:

1. Adaptive learning: An ability to learn how to do tasks based on the data given for training or initial experience.
2. Self-Organisation: An (ANN) can create its own organisation or representation of the information it receives during learning time.
3. Real Time Operation: (ANN) computations may be carried out in parallel, and special hardware devices are being designed and manufactured which take advantage of this capability.
4. Pattern recognition is a powerful technique for harnessing the information in the data and generalizing about it. Neural nets learn to recognize the patterns which exist in the data set.
5. The system is developed through learning rather than programming. Neural nets teach themselves the patterns in the data.
6. Neural networks are flexible in a changing environment. Although neural networks may take some time to learn a sudden drastic change, they are excellent at adapting to constantly changing information.
7. Neural networks can build informative models whenever conventional approaches fail. Because neural networks can handle very complex interactions, they can easily model data which is too difficult to model with traditional approaches such as inferential statistics or programming logic.
8. Performance of neural networks is at least as good as classical statistical modelling, and better on most problems.

2.9.5 Disadvantages of ANN

Zhang *et al.* (1998) posit that (ANN) model building needs lengthy experimentation and tinkering which is a major drawback for the extensive use of the method in forecasting. Rather, fuzzy expert system Bakirtzis *et al*. (1995); Bataineh *et al*. (1996) and wavelet analysis Zhang *et al*. (1995); have been proposed as supplementary tools to ANNs. 49

Thus, depending on the application, ANN can be a very robust, adaptive and easy to use alternative tool for predictions.

2.10 Population Projection

There are four methods population forecasting of future population These are: - Arithmetic progressive method, Incremental increase method, Geometric progression method and Exponential growth rate methods.

1. Arithmetic progressive method

Arithmetic progressive method: the average rate of increase in population is assumed to be constant from decade to decade

2. Geometric increase method

This method is based on the assumption that the percentage increase in population from decade to decade remains constant. This is also known as logarithmic growth method or exponential growth method

3. Incremental increase method

This method is improvement over the above two methods. The average increase in the population is determined by the arithmetical method and this is added to the present population to find the population of the next decade Yitbarek (2018)

CHAPTER THREE

3.0 RESEARCH METHODOLOGY

Research methodology is divided into three (3) stages.

Stage 1 is Data Collection,

Stage 2 is Field Work and

Stage 3 Pressure Distribution and Model Development.

3.1 City Profile and Population of the Study Area

Minna, the state capital of Niger State, has an origin that dated to 1905. At that time, Minna became an important workstation for the railways during the construction of the Lagos–Kaduna rail link. It is in the central part of Nigeria between latitude $9^{\circ}37'$ N to latitude $9^{\circ}40'$ N and longitude $6^{\circ}35'$ E to longitude $6^{\circ}39'$ E (Figure 3.1). The Minna economy is based mainly on trading and agricultural practices and has several educational institutions located within the city.

For purposes of this research the current population of the city, as calculated by United States Agency for International Development (USAID, 2020) E-WASH team, is 540,000 people with an average household size of nine members. Population was assigned to the loops based on the sizes of the area, taking into account the household size of nine (9) members. This was achieved by dividing the network into thirteen loops. Area of each loop was calculated. Average plot size of 450 m^2 was adopted. Figures 3.1 and 3.2 show the maps of Minna and Niger State repetitively. The pumping mains characteristics and their ages are depicted in Table 3.1. Minna water supply system is divided into eight pressure zones as indicated in Figure 3.3

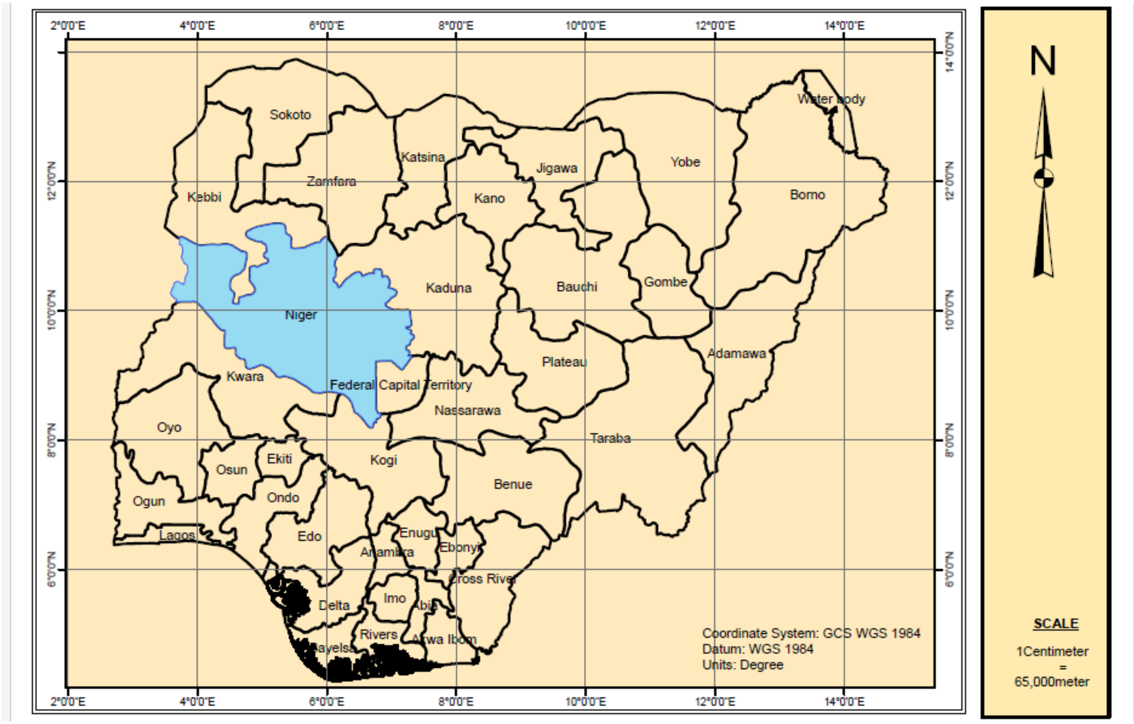


Figure 3.1: Niger State Map

Source: Federal Survey (1976)

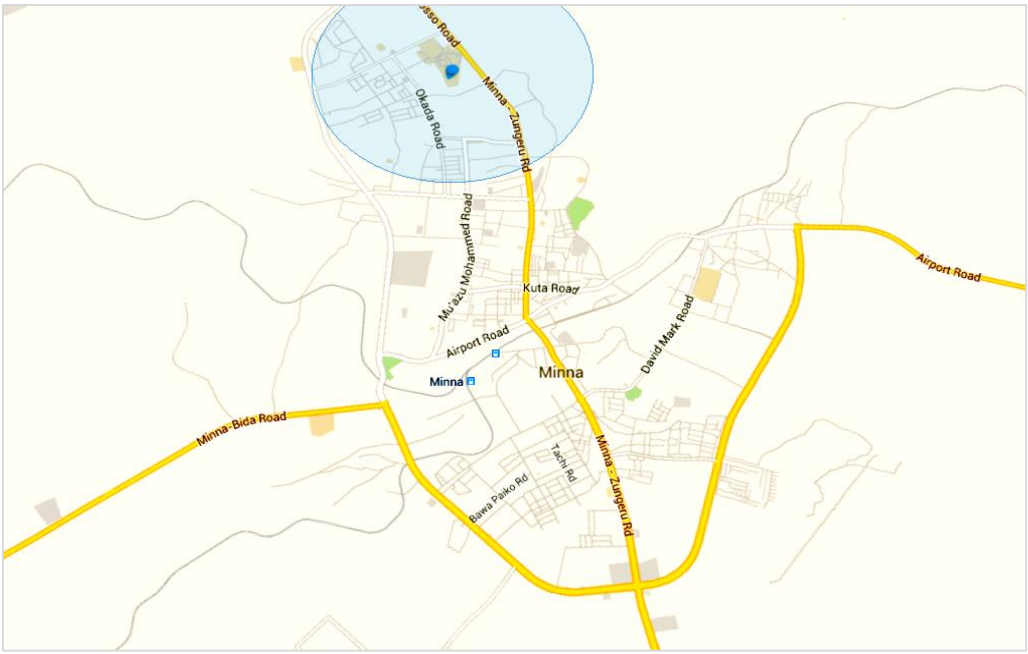


Figure 3.2: Minna city map showing the study Area

Source: Niger State Water and Sewage Corporation, (2019)

Table 3.1: Pumping Main Characteristics

Location	Diameter (DN mm)	Pipe Material/s	Age and current status
From Bosso Water Works to Bosso Town	100	Asbestos Cement	70 years; In good condition
From Paiko Water Works to Paiko Town	250	Asbestos Cement	25 years In Service
From Bi-Water Plant to Bi-Water Tank	300	Ductile Iron	Out of Service due to leakages on the mains
From Bi-Water Plant (27 mld) to INEC, IBB, Paida, and Bahago tanks (total	450	Ductile Iron	40 years In good condition
From Impresit (40 mld) to Top Medical, Bi-Water, Shiroro, and Dutsen Kura tanks (total capacity 18 mld)	700	Ductile Iron	35 years In good condition

Source: Niger State Water and Sewage Corporation (2019)

3.2 Distribution system

The distribution system characteristics of Minna Water Supply are depicted in Table 3.2

Eight pressure zones of Minna Water Distribution System is as shown in Figure 3.3.

Table 3.2: Details of mains in the Distribution System

Diameter (DN mm)	Pipe Material	Length (meters)	Age and Current Status of the pipelines
200-300	Asbestos Cement	37,222	40years: In Poor Condition
75-100	Asbestos Cement	14,895	40 years: In Poor Condition
75-250	uPVC	98,548	15 years: In Service Condition
150-250	Ductile Iron	66,000	35 years: In Service Condition
Total		216,665	

Source: NISWASEC (2019)

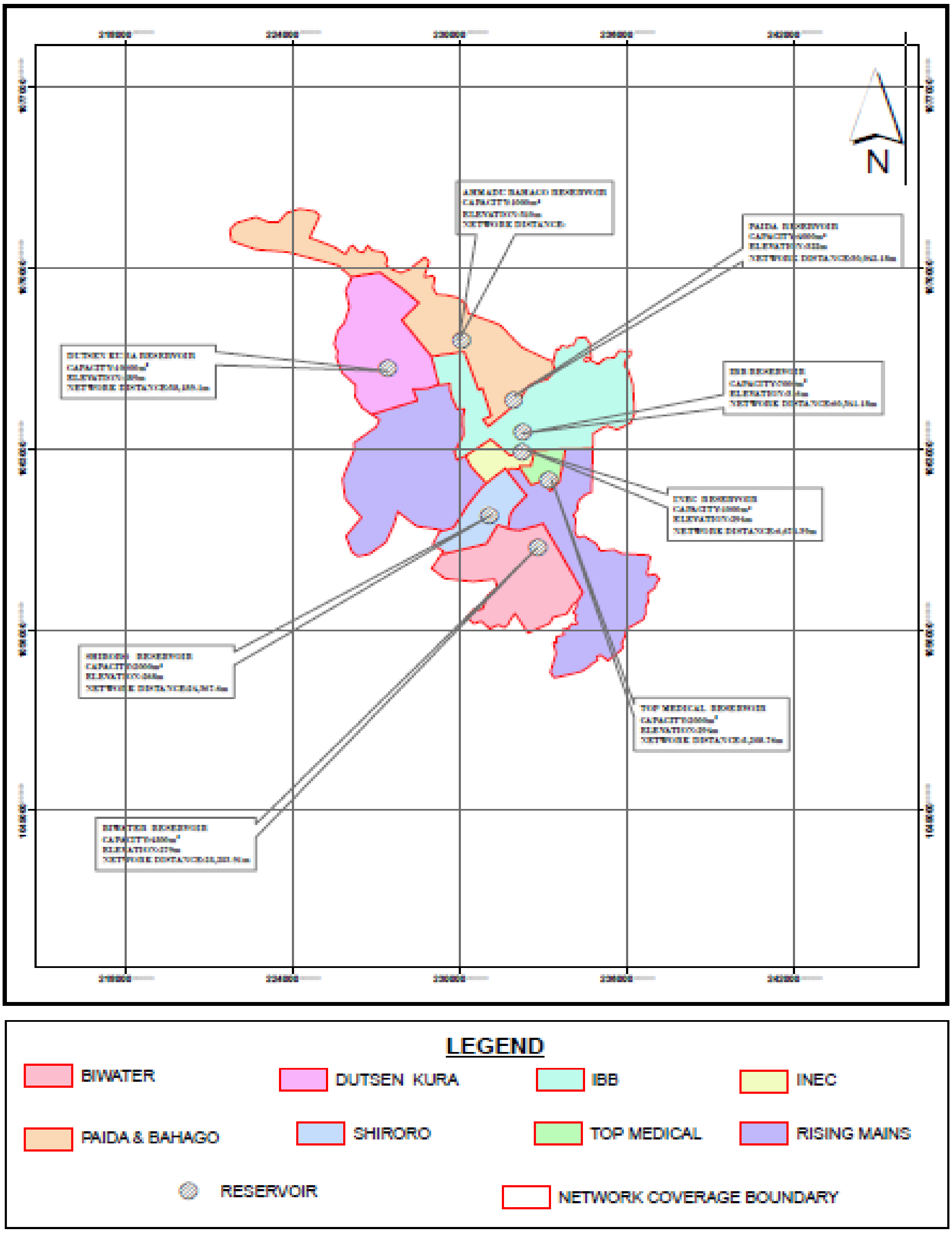


Figure 3.3: Minna Reservoir Coverage
Source: Niger State Water and Sewage Corporation, (2019)

3.3 Shiroro DMA

The imagery of the Shiroro Presssue zone and Shiroro DMA is indicated in Figure 3.4. The DMA has an estimated distribution line of 16,612 m of pipeline of which 11,500mare DN 150 mm uPVC pipes and DN 5112 m are DN 110 mm uPVC pipes.

The pumping main to Shiroro Tank is a DN 200 mm take-off on the pumping main from Impresit Plant at Chanchaga (40,000 m^3/d) to Dutsen Kura Reservoir. The pumping main is in good condition. In addition to Shiroro and Dutsen Kura Reservoir, the pumping main from the Impresit Plant also serves Top Medical and Bi-Water tanks. The total storage capacity fed by Impresit Plant is 18,000m^3 for the four service reservoirs.

Water from the Bi-Water Plant (27,000 m^3/d) feeds into INEC, IBB Tank, Paida, and Bahago tanks with a total storage capacity of 13,000 m^3. The pumping mains from both water treatment plants are interconnected. (NISWASEC, 2019)

Shiroro Reservoir has a concrete tank that can hold 2,000m^3 and originally received water from the Bi-Water tank with 4000m^3 capacity. Presently, Shiroro Tank receives water by a DN-200-mm take-off from a DN-700-mm pumping main to Dutsen Kura along the Western Bypass.

The sources of raw water for Minna Town are from Tagwai and Bosso Dams. While Bosso Dam was constructed in November 1949, Tagwai Dam was constructed in November 1978. The flow of raw water from Tagwai Dam to Chanchaga water treatment plant is by gravity along River Chanchaga. River water is directly drawn from River Chanchaga for water supply in Minna and serves as the main source of water for the town. The Tagwai Dam is in the city’s upstream (about 15 kilometres river length from the WTP location), and stored water is released into River Chanchaga. This dam was constructed in 1978, with designed storage capacity of 28.3 million cubic meters. Presently, there is no flow measurement option in the river.

There are two major water treatment plants next to River Chanchaga: Bi-water Water Treatment Plant (WTP) and Impresit (WTP). Details of Shiroro DMA is shownin Table 3.3. Figure 3.4 indicate the google earth View of the Shiroro pressure zone

Figure 3.4: Google Earth View of Shiroro Pressure Zone.
Source: NISWASEC (2019)

Table 3.3: Details pertaining to Shiroro Distribution Zone and Shiroro Model DMA

Aspect	Unit	Within Shiroro Distribution Zone	Within Shiroro Model DMA
Estimated length of water supply distribution lines	meters	16,600	14,000
Estimated land area	sq. km	5.17	1.9
Estimated population	nos.	23,445	19,998
Estimated number of households	nos.	2,605	2,222
Estimated existing water connection (active only)	nos.	345	240
Estimated future connections (considering inactive + new connections)	nos.	2,260	1,982
Estimated water demand (calculated @ 100 lpcd & 9 members per household)	kl/d	2,344.5	1,999.8
Estimated road network length	km	20.4	18
Estimated existing distribution line network	km	16.6 12	14

Source: NISWASEC (2019)

Shiroro's main water source is from a DN-700-mm rising main from Impresit Plant at Chanchaga Water Works, Minna. The take-off to the reservoir is through a DN-200-mm D.I. pipeline. Though the tank has 2000m^3 capacity, the tank's received volume is 2,713m^3 daily. Water quality is good. This storage reservoir can serve 20,000 people/day, assuming 100 lpcd. The carrying capacity of D 200 mm is 2,71 3m^3/d, assuming the pipeline's velocity of 1 m/sec, not taking into account frictional losses as the pipeline is less than one km (NISWASEC, 2019)

3.3.1 Methodology for evaluation of water supply network

Minna Water Supply System has Three Water Treatment Plants. These are Impresit, Biwater and Costain.Figure 3.5 shows the network layout of Minna water distribution system

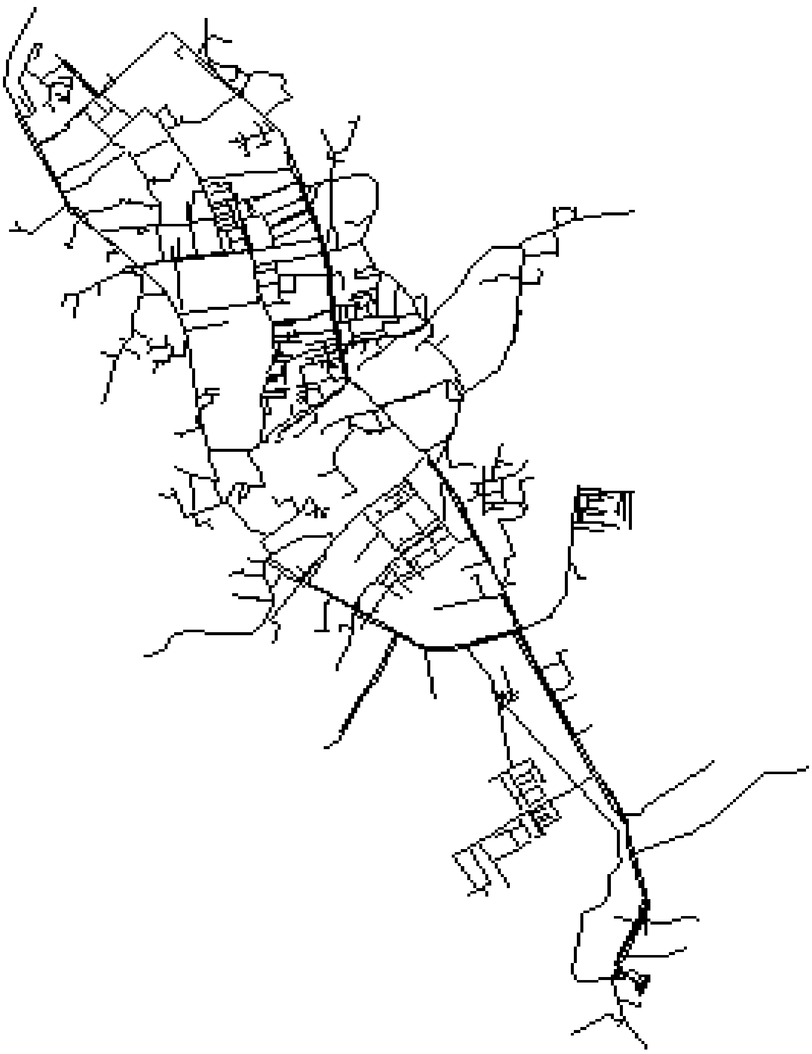


Figure 3.5: Network layout of Minna water distribution system

Visits were made to all tank sites to measure their diameters for loading. EPANET models diameter of tanks given initial, minimum and maximum levels. Diameter of a cylindrical tank is obtained by multiplying 1.128*square root of the cross-sectional area,

$$1.128 * \sqrt{L \times b}\ area\ or\ diameter\ D\ = 2 * \sqrt{\frac{l \times b}{\pi}} \qquad (3.1).$$

Niger State E WASH team has established that the total population of Minna is about 540,000 people which include the extended part of the city not covered by the existing network. This study established the total population of 429,957 people. Based on what is contained in the State WASH policy, which is nine person per household and 100LCPD, water demand was calculated. Institutional, commercial and losses constituted 20% of the domestic demand.

First stage digitization of the system was complete having over 800 junctions and 1000 links mapped out for model set up. Fixing of the attributes for links and nodes done successfully and the next stage was modelling and optimisation

Based on the policy adopted by Niger State WASH Policy (2020), the number of people per household is nine and per capita consumption is 100 liters per capita per day. At the current population of 429,957, the required domestic consumption is 54 million liters per day. The designed capacity of the plants in Minna is 70 million liters per day. This implies that at optimal production the existing plants in Minna can serve the population adopting100 LCPD as contained in the Niger State WASH Policy, (2020). Five (5%) of the total demand will be used for minor losses from bends, valves and tees, 15% for other factors like industrial, commercial, fire consideration. There is no data for residential occupants, nine (9) persons per household also agreed in base demand calculation as contained in the State Policy on water supply as well.

$$P_f = P_c \ (1 + r)^n \qquad (3.2)$$

when P_f = Future population

P_c = current population

n = design period

r= growth rate.

3.3.2 Network hydraulic modelling

The hydraulic machine, EPANET was used for the hydraulic modelling of the networks Other software machines employed are for data collection to accomplish this assignment include: ArcGIS, AutoCAD, EpaCAD, Google Earth Pro and TCX converter.

Shapefiles (using ArcGIS) from map/digitized map of transmission and distribution mains, reservoirs, tanks and valves were loaded to AutoCAD all geo referenced. These shape files loaded into AutoCAD were converted to metafile and used as backdrop in EPANET. The simulated backdrops were saved as NET File or INP file in EPANET interface.

The shapefiles were as well converted to (KML) and superimposed in google earth to obtain nodal elevation values. The shapefiles were equally loaded in AutoCAD and then converted to DXF file for terrain extractor to assign the nodal elevation values as check for nodal values. TCX converter utilized as well to verify correctness of key nodal point values which were viewed in excel sheet. The software EpaCAD was used to load AutoCAD directly to EPANET as INP file. Networks were then modelled.

Comprehensive methodologies analyses were carried out. Geo referenced network maps successfully loaded on to EPANET interface

EPANET network modelling tools perform real-time simulation of hydraulic behaviour within pipe networks and are designed to be a research tool that improves understanding of the movement and performance of water within a distribution system. An extended period simulation of 24 hours of the day was performed. Based on universal standards, a minimum pressure of 15m and maximum of 70m as well as a minimum flow-rate of 0.15 litres per second (LPS) were benchmarked. Though these pressure values vary from State to State. Locations with pressures and flow-rates that fell short of the standards were identified.

To simulate daily water use, analyse the pressure at each node and track the flow of water in each pipe, attribute data of the (EWTs), pipes and junctions were assigned into the software. The ground elevation of these components is very important in the network as it greatly determines the pressure and flow rate of water to homes, especially as the NISWASEC uses a pump fed distribution system. Parameters set included the Hazen-Williams (H-W) head-loss formula to compute the hydraulic head lost by water flowing in a pipe due to friction with the pipe walls (Almasri, 2012). Furthermore, a household water demand time pattern was assigned for each junction, representing demand at different times of the day. The 24-hour time period pattern was used, where water demand was modelled to be higher in the early hours of the morning and in the evening guided by Rossman's (2000) EPANET 2 User's Manual. This pattern also aided in Extended Period Simulation (EPS) of pressure and flow rate at different hours of the day (6:00am, 7:00, 8:00, 12 noon and 6:00pm).

3.4 Methodology for Leak Detection

FISHER H SCOPE XLT-17 Liquid Leak Detector Plate I and AML PRO Pipe Locator, Plate II, were used to identify the leaking sections of the pipeline. Hand held GPS, etrex 10, Plate III, was used to take the coordinates of sections of the pipeline. The **AML** Pro™ uses ultra-high radio frequencies to find buried PVC/PE pipes. This offers the best method for locating PVC & PE pipes and nearly any other pipes buried on ground.

The **Fisher XLT-17** is a liquid leak detector that finds leaks in four easy steps: by electronically amplifying leak sounds, selectively filtering out noise,

The etrex® 10 (or Garmin Oregon ® 600t) allows for the transfer of waypoint and track data between the GPS and the computer...

DR 300 pocket colorimeter is a handheld machine for the measurement of chlorine residual in WDS Calibrated buckets were used for the measurement of leak rates.

The layout where leakage detection was carried out was depicted in Figure 3.6

Figure 3.6: Leakage detection points

Plate I: AMLPRO Pipe Locator

Plate II: FISHER HSCOPE XLT -17 Liquid Leak Detector

Plate III: Handheld GARMIN eTrex 10 GPS

Plate IV: DR 300 Potable Colorimeter

This exercise was carried out for a period of 30 days. Nodal points with high pressure values as identified from the results of the simulation of the existing DMA were checked using the leak detecting equipment. The pictures of the field work are depicted in Plates V and VI.

3.5 Methodology for Pressure and Loss Relationship

Twenty four hours (24hrs) Extended Period Simulation, (EPS) was performed on M.I. Wushishi Water Distribution to evaluate the performance with respect to pressure and leakages. Figure 3.7 indicates the layout of the area where pressure and leakage relationship was carried out.

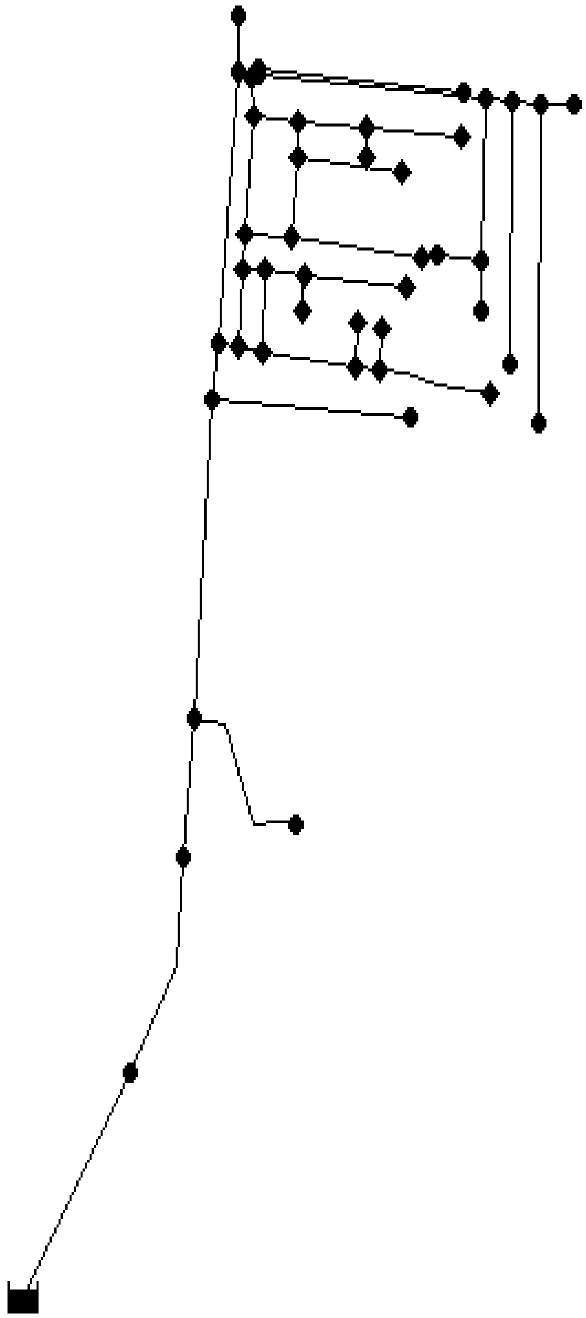


Figure 3.7: Layout of M.I. Wushishi Distribution Network (where pressure and leakage relationship was carried out)

3.5.1 Pressure and leak values

Pressure and leakage values of the model and observed were collated for correlation using statistical analysis to find out the degree of significance

3.6 Methodology for Model Parameter Calibration

Nash-Sutcliffe simulation efficiency (ENS) indicates how well the plot of observed versus simulated value fits the 1:1 line. The Nash–Sutcliffe efficiency is calculated as one minus the ratio of the error variance of the modelled time-series divided by the variance of the observed time-series.

$$E = 1 - \{ \sum_{t=1}^{T} (Q_o^t - Q_m^t)^2 / \sum_{t=1}^{T} (Q_o^t - \bar{Q})^2 \} \qquad (3.3)$$

Where;

Q_o = mean of observed discharges, and

Q_m = modelled discharge and

Qo^t = observed discharge at time t.

Nash–Sutcliffe efficiency ranges from infinity to 1. The value of efficiency of 1 (when E = 1) means there is a perfect match of modelled discharge relative to the observed data. The value of efficiency equal to (when E = 0) shows that the predictions of model are as accurate as the mean of the observed data, whereas an efficiency below zero (E < 0) occurs when the observed mean is a better predictor than the model or, in other words, when the residual variance (numerator in equation 3.3), is larger than the data variance (the denominator). Therefore, the closer the model efficiency is to 1, the more accurate the model is (Karthikeyan *et al.*, 2013). And according to Dongquan *et al.* (2009), an E_{NS} greater than 0.5 indicates acceptable model performance for model simulation.

Neural network construction predicts the independent variable giving the available information of independent variables, Neural networks are made up of a series of layers with each layer comprising at least one neuron. While intermediate layers (hidden layers) perform the data, processing functions of the network, the first and last layers input and output variables respectively. Within the hidden layers, weights to the neurons are adjusted by training the network in accordance with the stipulated learning rule (Zealand *et al.*, 1999). The neuron transfer function plays the role of transforming the input to output for each neuron. Typical neural network architecture is shown in Figure 3.8.

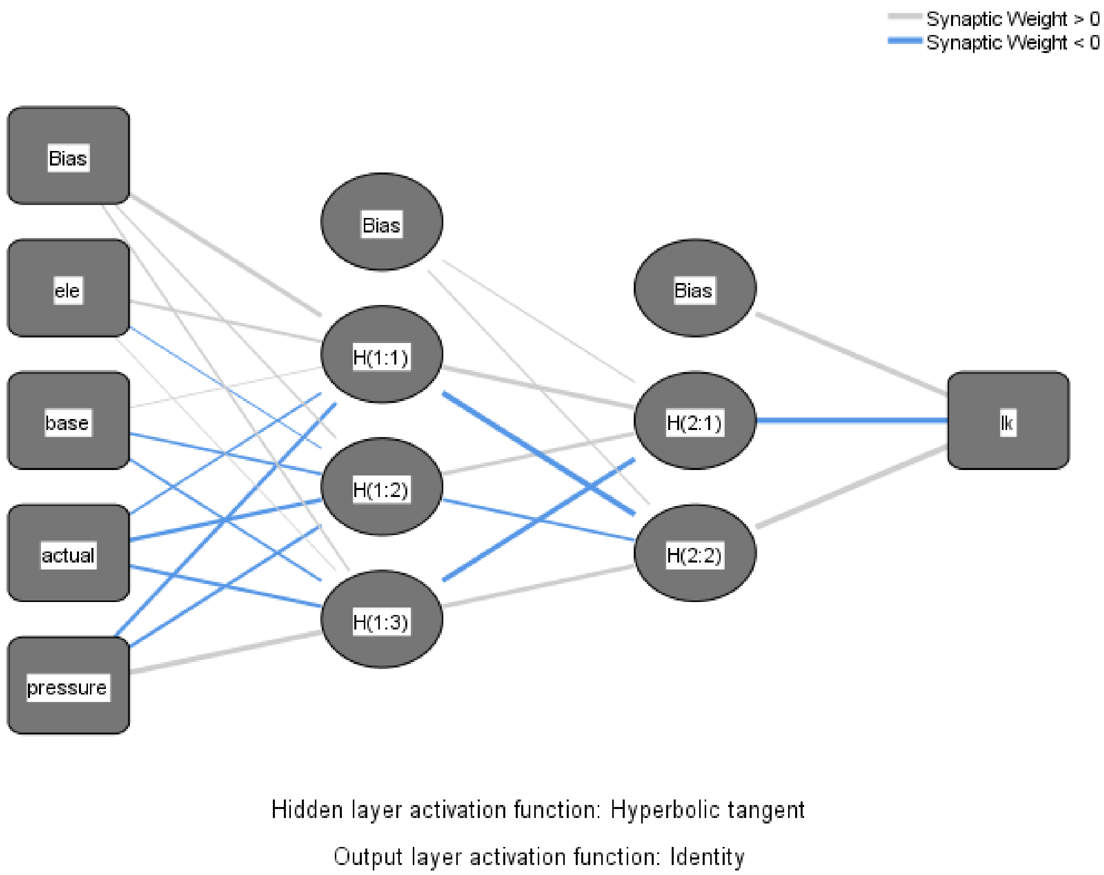


Figure 3.8: Neural Network Diagram

The input variables are elevation, demand and pressure while the output layer is the leakage which is dependent variable.

Emitter equation was used to simulate leakages at nodes. This is given by the equation

$$Q = a * P^b \tag{3.4}$$

Where Q = leakage (Q_{leak}), a and b are discharge coefficient and emitter exponent respectively and P is the pressure at the node.

ANN structure has three layers. Input Layer Information from the outside world enters the artificial neural network from the input layer. Input nodes process the data, analyse or categorize it, and pass it on to the next layer. The input variables are elevation, demand and pressure. Hidden layers take their input from the input layer or other hidden layers. Artificial neural networks can have a large number of hidden layers. Each hidden layer analyses the output from the previous layer, processes it further, and passes it on to the next layer. The output layer gives the final result of all the data processing by the artificial neural network. It can have single or multiple nodes. The output layer is the leakage

The Methodology framework for the leak prediction as shown in Figure 3.9

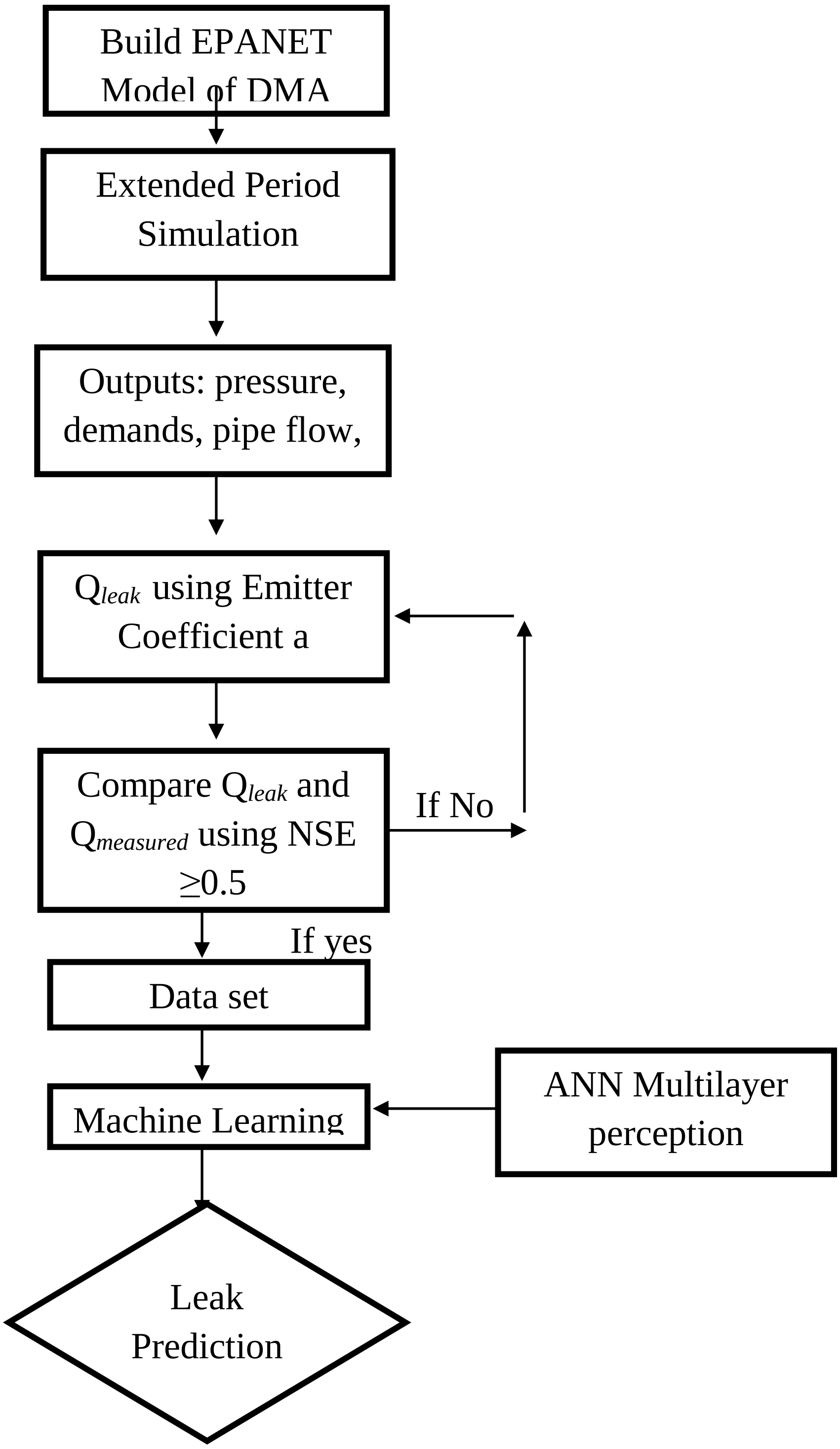


Figure 3.9: Methodology framework of leak prediction using ANN

In building the network, the author specified the number of hidden layers, neurons in each layer, transfer function in each layer, training function, weight/bias learning function, and performance function. The development of optimal network architecture was done using the graphical user interface of (SPSS), and validation of the water Loss.

For the learning algorithm, the feed-forward back-propagation algorithm was used. Regarding the transfer functions, hyperbolic tangent transfer function was used. The output layer neuron however used purelin transfer function so that the outputs can take any value between negative and positive infinity meaning that no scaling are needed on the outputs.

3.6.1 Leak collection at nodes

Rule based leakage identification: Section of the DMA with old pipes, smaller diameter, longer lengths of link, more service connections and sections of the network with residual chlorine lower than 0.1mg/l were identified as leak points with high probability. In obtaining the parameters of leakages for analysis in neural network, water samples were taken from different points in the DMA and analysed using pocket colorimeter DR300 and the reagents DPD procured in the course of this work. Samples with values of less than 0.1ppm or mg/l were carved out for leak modelling. Fields leak measurement are depicted in plates V and VI

Plate V: Leakage Identification

Plates VI: Field Leakage Measurement

Summary of steps used in leak modelling and estimation

Step 1: $Q = a * P^b$ was applied to nodes in the loop or (DMA) to estimate the leak, Rossman, (2000) and Burrows *et al., (*2003).

Q =Leakage, a = leakage coefficient and b = leakage exponent

To obtain Q_{leak}, nodes with the following conditions were considered to evaluate "a"

1 Nodes with values with residual chlorine less than 0.1mg

2 Nodes between aged Pipes

3 Nodes between longer length of Pipes >50m

4 Nodes with pipes having more service connections.

0.5 is used as leak exponent in as default in EPANET for pipes. Leak coefficient of 0.1, 0.15, 0.2 and 0.3 were varied to the nodes in this work.

Step 2: Model is run using logging data in (Appendix A) and values of Q_{leak} were generated (Appendix B1). Taking note that pressure head at each node was known after Extended Period Simulation of 24 hours

Step 3: The leaks on the 37 nodal demand points were physically measured using calibrated plastic containers, hoses, GPS, stop watch and flow meters. The values are indicated in Appendix B1

Step 4: The observed and the model values of the leaks were loaded onto the NASH Sutcliffe Efficiency Coefficient model to indicate how well the plot of the observed and modelled data fits the 1:1-line NSE = 1 which corresponds to a perfect match of the modelled to the observed data.

Step 5: Decision taken based on the NASH coefficients values.

Step 6: Elevation, Base demand, Demand and Pressure values of the simulation were used as independent variable to estimate leak using (SPSS) for (ANN) analyses. R^2 and Sum of square errors generated as output of Model

Step 7: 70% of the data set used for training and 30% data set was used for testing. (data generated from step 6). In network building several configurations were tried and the one with the “best” prediction efficiency was chosen to be the network training and testing. The weights were adjusted in order to make the actual outputs (predicated) close to the target (measured) outputs of the network. In this study, pressure, elevation, base demand, demand, head were used as the input data and the loss/leakage was the output data

The next step was to test the performance of the developed (ANN) model. Data collated from the DMA was used. In order to evaluate the performance of the developed (ANN) model quantitatively and verify whether there is any underlying trend in performance of ANN model, statistical analysis involving the coefficient of determination (R^2), and the root mean square error (RMSE) was computed. RMSE provides information on the short-term performance which is of the variation of predicted values around the measured area. The values of the predictors obtained were divided into three. 70% of the first two was used for training and 30% was used for testing in Artificial Neural Network multi-layer perception. The model was optimized by using hyperbolic tangent. Figure 3.10 depicts the network modelling of the DMA in 24 hours Extended Period Simulation and details of the input data in Appendix A

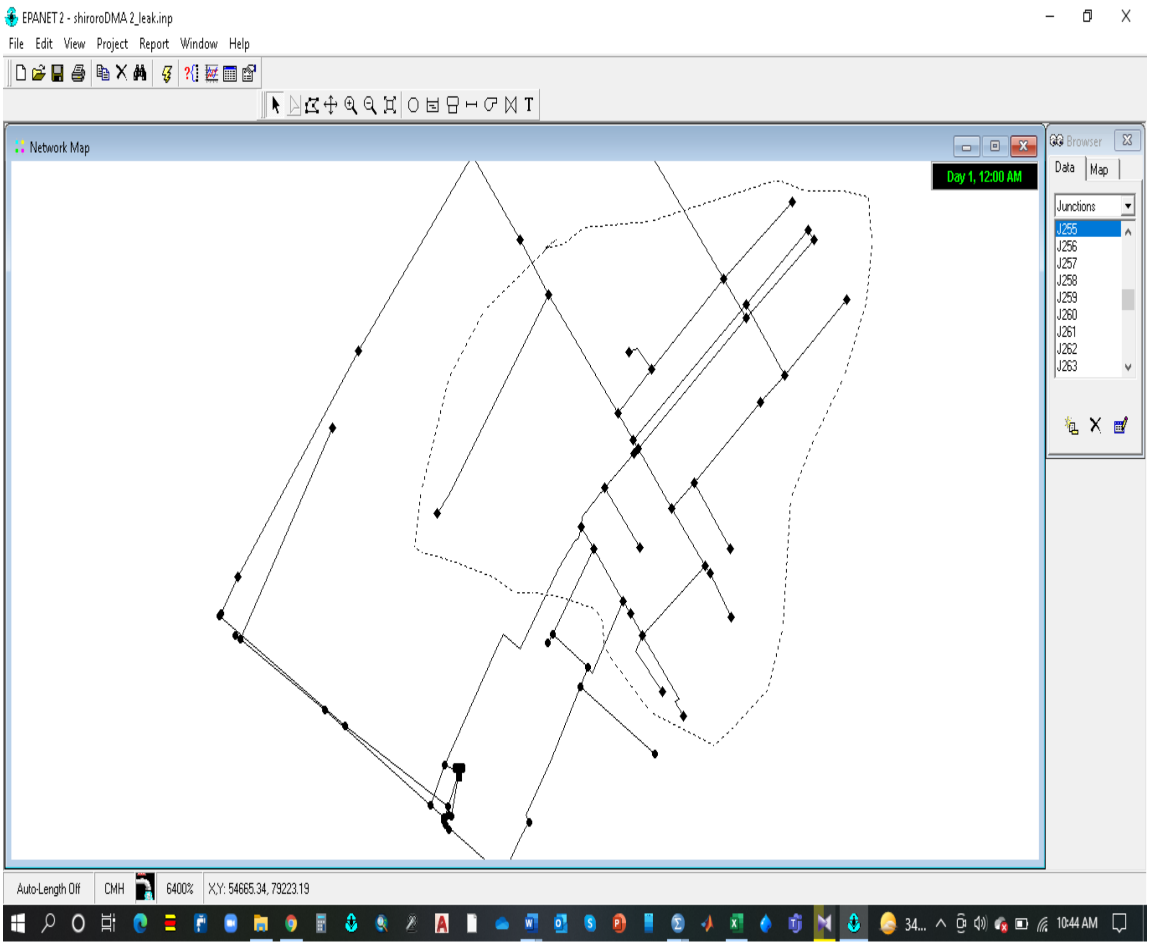


Figure 3.10: EPANET interface of Shiroro DMA showing the selected nodes for analysis (allow EPANET to distinguish nodes, rather plane dots)

CHAPTER FOUR

4.0 RESULTS AND DISCUSSION

4.1 Description of the Network

Biwater/Costain Company of U.K. were the first to carry out project of expanding water works between 1978/79 and 1986. Reticulation of some parts of Minna network was also carried out. Most asbestos and steel pipes in old Minna are were laid by the company. The second project was executed by Impresit Bakolori of Italy in 1995. During this time Police Secondary School tank which is the largest in Minna was constructed, its capacity is 10,000$m^{3.}$ Top medical tank and Paida tank were also constructed by this company. The total distribution network coverage is 216,665m and transmission main is 75,000m. There are eight service reservoirs supplying the town situated at strategic points. This is indicated in Table 4.6.

4.2 Result for Evaluation of Water Supply Network

The pumps characteristics of Chanchaga Water Works is shown in Table 4.1. Table 4.2 shows reservoir features of Minna Water Distribution System

Table 41: The pumps at the headwork with their parameters

S/N	Pump	Q (m^3/h)	Head (m)
1.	KSB 1	558	150
2.	KSB 2	558	150
3.	KSB 3	558	150
4.	KSB 4	558	150
5.	KSB 5	558	150
6.	KSB 6	558	150
7.	KSB 7	558	150
8.	KSB 8	558	150
9.	KSB 9	558	150
10.	KSB 10	558	150

Table 4.2: **The service tanks and their details**

S/N	Tank	Vol (m^3)	Dia (m)	Elevation (m)	Tank Type
1.	Dutsen Kura	10,000	44	289	Concrete
2.	IBB	7,000	38	316	Concrete
3.	Biwater	4,500	36	249	Braithwaite
4.	Paida	4,000	28	322	Concrete
5.	Shiroro	2,000	24	268	Concrete
6.	Top Medical	2,000	25	294	Concrete
7.	INEC	1,000	22	284	Concrete
8.	Bahago	1,000	18	310	Concrete

4.2.1 Population and demand analysis

With current population of 429,957, growth rate 3.5, geometric projection technique was used to project population to 2030 as 720,327. Table 4.3 show the population distribution per loop and water demand in LCPD Figure 4.1 show the Layout of the loop of CCE at M. I. Wushishi Water Distribution Network. The loops of WDS is in Appendix D

Figure 4.1: Layout of M. I. Wushishi Water Distribution Network

Table 4.3: Current Peak Demand Table

	Loop	Area (Sqm)	Pop	Current Demand (m^3/h)	Current Peak Dem (m^3/h)	Future PeakDem (m^3/h)	Location
1	CCA	970,425	13590	67.95	95.13	113.40	Army Barrack
2	CCB	1,960,184	22473	112.32	157.25	188.00	Mandela
3	CCC	571,386	7992	39.96	55.92	66.94	Tunga
4	CCD	3,420,411	47880	239.42	335.24	401.12	Tunga
5	CCE	478,769	6786	33.48	46.92	50.15	M.I Wushishi
6	CCF	947,255	13,266	66.24	92.73	110.00	Kafin Tela
7	CCG	3,769,378	52,767	263.88	369.41	442.00	Police Barrack
8	CCH	2,423,578	33,930	169.56	237.44	284.00	Airport Qtrs
9	CCI	3,864,825	54,099	270.36	378.51	452.00	GRA/D/kura
10	CCJ	2,570,413	35,982	180.12	252.00	300.01	Fadikpe
11	CCK	5,182,899	72,558	362.88	508.00	607.00	Bosso Low Cost
12	CCL	1,165,382	16,308	81.72	114.43	136.00	Bosso Estate
13	CCM	3,746,661	52,407	262.08	366.90	439.00	Tayi/F-layout

Graphical representation of current and future demand are shown in Figure 4.2.

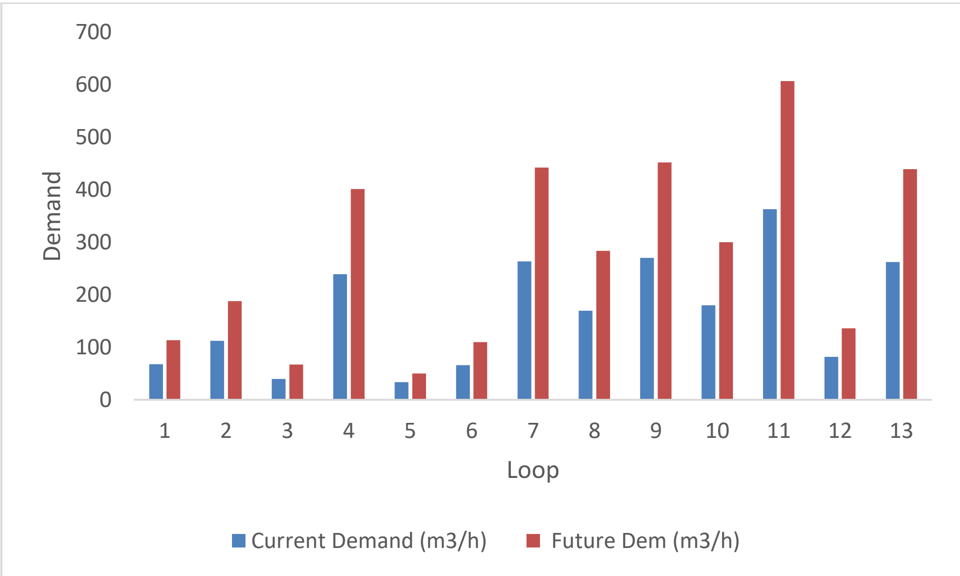


Figure 4.2: Current and future demand in Minna Pls distinguish axes

4.2.2 Minna WTP main network

Single period Analyses and Extended Period Simulations were carried out for an instant and extended period of 24 hours results. There are flows in all the pipes but the values are dependent on the degree of accuracy set.

Networks were simulated in order to evaluate the performance of the with respect to pressure, flow and head loss. The major outputs are the pressure values at demand nodes. The nodes with positive demands required positive values of pressure Minna WTP had 892 nodes and 1021 links

First run showed warning messages, with many negative pressures at demand point, 17 pipes out of 1,021 were altered in term of diameter increase and a successful run was achieved. The calibration is as depicted in Table 4.4.

Table 4.4: The original pipe size and calibrated diameter for steady state simulation

S/N	Pipe	Existing Diameter (mm)	New Diameter (mm)
1.	902	100	200
2.	969	100	200
3.	976	100	200
4.	977	100	200
5.	978	100	200
6.	979	100	200
7.	980	100	200
8.	971	100	200
9.	984	100	200
10.	981	100	200
11.	983	100	200
12.	982	100	200
13.	973	100	200
14.	974	100	200
15.	975	100	200
16.	972	100	200
17.	970	100	200

The main pipe supplying the pipes from Bahago Reservoir is modified from 100mm to 300mm. High pressure noticed along Army barrack, as a result pressure reducing valve, PRV was modelled to bring down high pressure which may eventually cause the breakage of pipes. The system has the pumps with capacity to supply the network satisfactorily but inefficient due to correct size of same pipes. Table 4.5 indicates single period analysis of Minna (WTP).

Table 4.5: The Single Period Analysis Minna WTP

Node ID	Elevation (m)	Base Demand (m^3/h)	Demand (m^3/h)	Head (m)	Pressure (m)
Junc J4	222	0	0	412.93	190.93
Junc J1	217	0	0	324.09	107.09
Junc J5	221	0	0	412.91	191.91
Junc J6	227	0	0	357.54	130.54
Junc J7	246	0	0	336.94	90.94
Junc J10	247	3.62	3.62	282.54	35.54
Junc J11	250	3.57	3.57	296.19	46.19
Junc J12	280	3.57	3.57	289.66	9.66
Junc J13	281	0	0	317.28	36.28
Junc J14	279	3.14	3.14	311.18	32.18
Junc J15	250	3.86	3.86	276.23	26.23
Junc J16	239	3.57	3.57	285.7	46.7
Junc J17	247	3.14	3.14	289.31	42.31
Junc J18	262	0	0	328.19	66.19
Junc J19	265	4.6	4.6	321.94	56.94
Junc J21	264	0	0	318.52	54.52
Junc J22	255	0	0	306.67	51.67
Junc J9	221	0	0	372.85	151.85
Junc J20	225	0	0	367.94	142.94
Junc J23	224	0	0	367.77	143.77
Junc J24	219	3.86	3.86	367.39	148.39
Junc J25	225	3.86	3.86	367.69	142.69
Junc J26	213	3.86	3.86	372.57	159.57
Junc J27	227	0	0	359.98	132.98
Junc J28	227	0	0	367.7	140.7
Junc J29	234	0.82	0.82	367.66	133.66
Junc J877	288	4.84	4.84	311.45	23.45
Junc J878	280	3.14	3.14	311.04	31.04
Tank Biwater	279	N/A	566.21	281.5	2.5

Table 4.6: Single Period Analysis Link Result Minna WTP

Link ID	Length (m)	Diameter (mm)	Roughness	Flow (m^3/h)	Velocity (m/s)	Unit Headloss (m/km)
Pipe P2	2550.42	300	130	602.88	2.37	16.7
Pipe P6	1164.61	900	130	314.16	0.14	0.02
PipP18	551.22	450	130	1455.61	2.54	11.86
PipeP17	1000	450	130	1304.07	2.28	9.67
PipeP13	916.52	100	130	3.86	0.14	0.3
PipeP14	587.29	450	130	1205.37	2.11	8.36
PipeP15	835.56	100	130	3.86	0.14	0.3
PipeP16	20.6	450	130	1201.51	2.1	8.31
PipeP21	308.54	75	130	3.86	0.24	1.24
PipeP22	943.23	450	130	1197.65	2.09	8.26
PipeP24	250.53	450	130	1308.17	2.28	9.73
PipeP25	332.42	450	130	1308.17	2.28	9.73
PipeP26	1325.64	450	130	1307.35	2.28	9.72
PipeP27	278	150	130	0.82	0.01	0
PipeP28	533.61	450	130	1306.53	2.28	9.71
PipeP29	322.52	100	130	0.82	0.03	0.02
PipeP30	543.88	450	130	1305.71	2.28	9.69
PipeP31	343.4	100	130	1.64	0.06	0.06

For first run of extended period simulation, many junctions were disconnected from the system. A scenario was created to link top medical and INEC tanks through with the pipe which eliminate disconnected junction messages. However, the network was still unstable due to negative pressure values at the demand points

Another scenario was developed to further stabilize the system which is modification of some tanks in the system. The height of Paida was raised to 11m. Dutsen kura 11m, and Bahago 9 m. A successful run of extended period simulation was achieved. This is clearly depicted in Table 4.7. Table 4.8 shows the summary of EPS of Minna WTP

Table 4.7: The modifications to the tanks to have all demand supplying water for a period of 24 hours

S/N	Tank	Initial Height (m)	Maximum Height (m)	Diameter (mm)
1.	Paida	9	11	60
2.	Dutsen Kura	9	11	60
3.	Bahago	7	9	45

Table 4.8: Summary of EPS Minna WTP

Node ID	Elevation (m)	Base Demand (m3/h)	Demand (m3/h)	Head (m)	Pressure (m)
Junc J4	216	0	0	413.46	197.46
Junc J1	217	0	0	331.24	114.24
Junc J5	221	0	0	413.44	192.44
Junc J6	227	0	0	377.37	150.37
Junc J7	246	0	0	366.59	120.59
JuncJ10	247	1.81	2.17	308.21	61.21
JuncJ11	250	1.79	2.15	344.62	94.62
JuncJ12	280	1.79	2.15	344.62	64.62
JuncJ13	281	0	0	322.95	41.95
JuncJ14	279	1.6	1.92	319.14	40.14
JuncJ15	250	1.9	2.28	307.71	57.71
JuncJ16	239	1.79	2.15	306.67	67.67
JuncJ17	247	1.6	1.92	306.67	59.67
JuncJ18	262	0	0	360.71	98.71

4.2.3 Peak scenario of Minna WTP

When the system was run for the peak scenario, warning messages were generated in junctions 379, 380, 381, 382, 383, 384, 385, 446, 518, 600, 601 and 605. Table 4.9 portrays the modification for peak horizon. Link results of the peak scenario is indicated in Table 4.10

Table 4.9: Modification for the eliminate of negative pressures in the peak scenario.

S/N	Pipe	Existing Diameter (mm)	New Diameter (mm)
1.	648	100	300
2.	545	100	150
3.	473	100	300
4.	474	100	225
5.	478	100	200

Table 4.10: Minna WTP Node Peak Results

	Elevation m	Base Demand (m^3/h)	Demand (m^3/h)	Head (m)	Pressure (m)
Junc J4	216	0	0	412.65	196.65
Junc J1	217	0	0	329.8	112.8
Junc J5	221	0	0	412.62	191.62
Junc J6	227	0	0	320.92	93.92
Junc J7	246	0	0	324.19	78.19
Junc J10	247	5.1	5.1	281.47	34.47
Junc J11	0	5	5	292.61	292.61
Junc J12	280	5	5	288.94	8.94
Junc J13	281	0	0	316.34	35.34
Junc J14	279	4.4	4.4	309.95	30.95
Junc J15	0	5.4	5.4	274.67	274.67
Junc J16	239	5	5	284.02	45.02
Junc J17	247	4.4	4.4	288.41	41.41
Junc J18	262	0	0	307.24	45.24
Junc J19	265	6.4	6.4	321.47	56.47
Junc J21	264	0	0	302.75	38.75
Junc J25	225	0	0	384.18	159.18
Junc J26	213	0	0	386.69	173.69
Junc J27	227	0	0	380.06	153.06
Junc J28	227	0	0	380.7	153.7
Junc J29	234	75	75	235.89	1.89
Junc J30	220	0	0	319.41	99.41
Junc J31	247	1.1	1.1	319.23	72.23
Junc J32	245	0	0	313.4	68.4
Junc J62	273	2.8	2.8	298.96	25.96
Junc J63	264	0	0	302.74	38.74

4.2.4 Future scenario

A scenario was made for projection of 15 years from now using the existing system.

The run showed unstable network due to negative pressure at 28 demand points representing 3.1% and these include junctions 379, 380, 381, 382, 383, 385, 386, 513, 514, 515, 516, 517, 518, 519, 520, 521, 522, 523, 524, 525, 599, 600, 601, 602, 603, 604, 605. Table 4.11 illustrates the calibration for future horizon. Node values of future scenario is indicated in Table 4.12.

Table 4.11: Calibration carried out for the elimination negative pressure in the future horizon

S/N	Pipe	Existing Diameter (mm)	New Diameter (mm)
1.	473	100	225
2.	474	100	200
3.	475	100	200
4.	476	100	200
5.	478	75	225
6.	479	75	200
7.	484	75	200
8.	545	100	225
9.	553	100	150
10.	546	100	150
11.	648	100	200

Table 4.12: Minna WTP Future Nodes Results

	Elevation (m)	Base Demand (m^3/h)	Demand (m^3/h)	Head (m)	Pressure (m)
Node ID	m	CMH	CMH	m	m
Junc J4	216	0	0	412.55	196.55
Junc J1	217	0	0	323.64	106.64
Junc J5	221	0	0	412.52	191.52
Junc J6	227	0	0	356.29	129.29
Junc J7	0	0	0	335.62	335.62
Junc J10	247	6.1	6.1	280.91	33.91
Junc J11	0	6	6	295.35	295.35
Junc J12	280	6	6	289.5	9.5
Junc J13	281	0	0	315.75	34.75
Junc J14	279	5.3	5.3	309.24	30.24
Junc J15	0	6.5	6.5	274.04	274.04
Junc J16	239	6	6	282.9	43.9
Junc J17	247	5.3	5.3	287.78	40.78
Junc J18	262	0	0	326.71	64.71
Junc J19	265	7.7	7.7	321.15	56.15
Junc J21	264	0	0	316.98	52.98
Junc J22	255	0	0	305.48	50.48
Junc J31	247	1.2	1.2	352.82	105.82
Junc J32	245	0	0	340.04	95.04
Junc J33	246	0	0	334.82	88.82
Junc J34	239	1.2	1.2	340.04	101.04
Junc J35	254	0	0	329.51	75.51
Junc J36	247	1.2	1.2	329.47	82.47
Junc J37	247	1.2	1.2	329.46	82.46
Junc J38	263	0	0	311.29	48.29
Junc J39	250	6	6	295.63	45.63
Junc J40	259	6	6	321.01	62.01
Junc J41	255	6	6	321.7	66.7
Junc J42	251	6	6	322.37	71.37
Junc J43	251	6	6	321.61	70.61
Junc J60	290	8.1	8.1	310.68	20.68

In Minna (WTP), 23 negative values at different joints out of 892 were optimised to make the network efficient. The analysis indicated 46% of nodes are deficient in pressure values at 11am. All negative values at the nodes were eliminated after optimisation

4.2.5 Analyses by water treatment plant

Minna Water Supply System is divided into three district networks i.e., Impresit Biwater and Costain which make the main network of Minna analyzed earlier. Further investigation on isolating the district networks was carried out

Initial run of Biwater Network showed many negative pressures at demand points. Modifications were carried out for this scenario to eliminate negativity and stabilize the network. Twenty One (21 %) of 81 nodes showed negative values of pressure

Table 4.13 depicts the modification done to eliminate negative pressure and stabilize the network in terms of flow and velocity in the pipe. Node results is indicated in Table 4.14. Peak values of nodes are shown in Table 4.15

Table 4.13: Biwater Network modification

S/N	Pipe	Existing Diameter (mm)	New Diameter (mm)
1.	11	100	225
2.	7	150	225
3.	25	100	225
4.	8	100	150
5.	9	100	150
6.	10	100	150
7.	11	100	150
8.	12	100	150
9.	13	100	150
10.	14	100	150
11.	15	100	150
12.	16	100	150
13.	17	100	150
14.	18	100	150
15.	19	100	150
16.	20	100	150
17.	21	100	150
18.	22	100	150
19.	23	100	150

Table 4.14: Biwater WTP SPA Node Results

	Elevation (m)	Base Demand (m)	Demand (m^3/h)	Head (m^3/h)	Pressure (m)
Junc J1	217	0	0	333.87	116.87
Junc J10	247	3.62	3.62	272.08	25.08
Junc J94	254	3.86	3.86	270.96	16.96
Junc J105	215	0	0	394.57	179.57
Junc J106	215	0	0	356.83	141.83
Junc J107	221	0	0	342.97	121.97
Junc J166	249	3.62	3.62	272.07	23.07
Junc J167	258	0	0	272.07	14.07
Junc J205	250	3.62	3.62	271	21
Junc J206	238	3.62	3.62	271.2	33.2
Junc J207	236	3.62	3.62	271.24	35.24
Junc J208	237	3.62	3.62	271.55	34.55
Junc J209	256	3.62	3.62	275.36	19.36
Junc J210	261	3.62	3.62	275.49	14.49
Junc J211	267	3.62	3.62	276.25	9.25
Junc J212	267	3.62	3.62	276.55	9.55
Junc J213	266	3.62	3.62	276.23	10.23
Junc J214	262	3.62	3.62	276.49	14.49
Junc J215	261	3.62	3.62	275.41	14.41
Junc J216	261	3.62	3.62	275.43	14.43
Junc J217	257	3.62	3.62	276.46	19.46
Junc J218	257	3.62	3.62	275.4	18.4
Junc J219	242	3.62	3.62	271.34	29.34
Junc J220	266	3.62	3.62	279.77	13.77
Junc J221	251	3.62	3.62	270.95	19.95
Junc J222	252	3.62	3.62	270.19	18.19
Junc J223	253	3.62	3.62	270.46	17.46
Junc J224	245	3.62	3.62	270.28	25.28
Junc J225	239	3.62	3.62	268.67	29.67
Junc J226	234	3.62	3.62	270.76	36.76
Junc J227	236	3.62	3.62	270.71	34.71
Junc J228	254	3.86	3.86	270.97	16.97
Junc J230	254	3.86	3.86	270.96	16.96
Junc J263	254	0.87	0.87	266.69	12.69

Table 4.15: Biwater WTP Peak Link

Link ID	Length (m)	Diameter (mm)	Roughness	Flow (m^3/h)	Velocity (m/s)	Unit Headloss (m/km)
Pipe P2	2550.42	300	130	674.09	2.65	20.53
Pipe P113	1838.24	300	130	674.09	2.65	20.53
Pipe P114	674.78	300	130	674.09	2.65	20.53
Pipe P115	443.1	300	130	674.09	2.65	20.53
Pipe P180	83.72	200	130	25.5	0.23	0.34
Pipe P181	476.33	225	130	0	0	0
Pipe P231	553.96	225	130	46.66	0.33	0.59
Pipe P232	400.41	225	130	36.46	0.25	0.38
Pipe P233	70.99	225	130	26.26	0.18	0.2
Pipe P234	463.63	225	130	16.06	0.11	0.08
Pipe P235	137.71	225	130	5.86	0.04	0.01
Pipe P237	757.15	100	130	5.1	0.18	0.51
Pipe P239	166.82	100	130	51	1.8	36.32
Pipe P240	30.28	100	130	35.7	1.26	18.76
Pipe P241	77.33	100	130	5.1	0.18	0.51
Pipe P242	143.89	100	130	25.5	0.9	10.06
Pipe P243	450.03	100	130	5.1	0.18	0.51
Pipe P244	27.48	100	130	15.3	0.54	3.91
Pipe P245	106.11	100	130	5.1	0.18	0.51
Pipe P246	73.03	100	130	-5.1	0.18	0.51
Pipe P247	160.49	100	130	6.14	0.22	0.72
Pipe P248	345.21	100	130	4.06	0.14	0.33
Pipe P249	137.23	100	130	5.1	0.18	0.51
Pipe P250	75.33	150	130	133.36	2.1	29.89
Pipe P251	728.42	150	130	77.26	1.21	10.88
Pipe P252	1952.41	100	130	5.1	0.18	0.51
Pipe P253	1632.84	100	130	5.1	0.18	0.51
Pipe P254	34.54	225	130	-14.84	0.1	0.07
Pipe P255	29.87	225	130	-76.49	0.53	1.48
Pipe P293	223.78	150	130	5.72	0.09	0.09

This (WTP) had 96 nodes and 111 links. 86% of the nodes showed pressure values within the bench mark of 15 to 15m

The first few runs of Costain steady state simulation recorded high pressures above 50m from Paida Tank. Adjustment was carried out by introduction of DN300mm PRV with setting 100. Further correction for Costain WTP SPA is as shown in Table 4.16. Pressure values are depicted in Table 4.17

Table 4.16: Modification for Costain WTP network

S/N	Pipe	Existing Día (mm)	Suggested Día (mm)
1.	329	100	150
2.	345	100	150
3.	334	100	150
4.	330	100	150
5.	331	100	150
6.	332	100	150
7.	333	100	150
8.	334	100	150
9.	339	100	150
10.	340	100	150
11.	337	150	100

Table 4.17a: Costain WTP SPA Node Table

	Elevation (m)	Base Demand (m^3/h)	Demand (m^3/h)	Head (m)	Pressure (m)
Junc J6	0	0	0	363.47	363.47
Junc J11	250	3.57	3.57	298.23	48.23
Junc J12	280	3.57	3.57	290.02	10.02
Junc J13	281	0	0	317.88	36.88
Junc J14	279	3.14	3.14	312.32	33.32
Junc J18	262	0	0	332.93	70.93
Junc J19	265	4.6	4.6	322.14	57.14
Junc J21	264	0	0	322.84	58.84
Junc J22	255	0	0	309.84	54.84
Junc J32	245	0	0	346.71	101.71
Junc J33	246	0	0	341.32	95.32
Junc J34	239	0.82	0.82	346.71	107.71
Junc J38	263	0	0	316.44	53.44
Junc J39	250	3.57	3.57	298.67	48.67
Junc J40	259	3.57	3.57	321.5	62.5
Junc J41	255	3.57	3.57	321.77	66.77

Table 4.17b: Costain WTP SPA Node Table

	Elevation (m)	Base Demand (m^3/h)	Demand (m^3/h)	Head (m)	Pressure (m)
Junc J42	251	3.57	3.57	322.44	71.44
Junc J43	251	3.57	3.57	322.15	71.15
Junc J44	268	1.18	1.18	324.3	56.3
Junc J45	261	1.18	1.18	324.05	63.05
Junc J46	270	1.18	1.18	323.32	53.32
Junc J47	269	1.18	1.18	323.4	54.4
Junc J48	271	4.6	4.6	322.56	51.56
Junc J49	269	4.6	4.6	322.46	53.46
Junc J50	266	4.6	4.6	322.3	56.3
Junc J51	263	4.6	4.6	321.65	58.65
Junc J52	272	4.6	4.6	321.09	49.09
Junc J53	266	4.6	4.6	320.34	54.34
Junc J54	273	4.6	4.6	314.05	41.05
Junc J55	273	4.6	4.6	314	41
Junc J56	274	4.6	4.6	313.92	39.92
Junc J80	282	0.19	0.19	295.8	13.8

When the Costain network was modelled for peak scenario, warning messages were generated as a result of negative pressures in junction 379, 380, 381, 382, 385, 600, 60, 605.equivalent to 2% of the total number of nodes

Table 4.18 indicates the modification done to have stable network with acceptable pressure valves at junctions and flow in pipes, while Table 4.19 indicates link results of peak horizon.

Table 4.18: Modification for peak Costain network

S/N	Pipe	Existing Diameter (mm)	New Diameter (mm)
1.	473	100	300
2.	478	75	200
3.	474	100	200
4.	648	100	225

Table 4.19: Costain WTP Peak Link Table

	Length (m)	Diameter (m)	Roughness	Flow (m^3/h)	Velocity (m/s)	Unit Headloss
Pipe P18	551.22	450	130	1599.14	2.79	14.11
Pipe P19	56.25	450	130	4285.68	7.49	87.59
Pipe P20	780.17	450	130	-2691.53	4.7	37.01
Pipe P17	1000	450	130	1337.52	2.34	10.14
Pipe P13	916.52	100	130	0	0	0
Pipe P14	587.29	450	130	1220.15	2.13	8.55
Pipe P15	835.56	100	130	0	0	0
Pipe P16	20.6	450	130	1220.15	2.13	8.55
Pipe P21	308.54	75	130	0	0	0
Pipe P22	943.23	450	130	1220.15	2.13	8.55
Pipe P23	50.78	100	130	-120.67	4.27	179
Pipe P24	250.53	450	130	1340.82	2.34	10.18
Pipe P25	332.42	450	130	1340.82	2.34	10.18
Pipe P26	1325.64	450	130	1339.72	2.34	10.17
Pipe P27	278	150	130	1.1	0.02	0
Pipe P28	533.61	450	130	1338.62	2.34	10.15
Pipe P29	322.52	100	130	1.1	0.04	0.03
Pipe P35	319.2	450	130	1283.18	2.24	9.39
Pipe P36	366.5	450	130	1279.58	2.23	9.34
Pipe P37	225.75	50	130	2.8	0.4	4.92
Pipe P38	706.92	450	130	1276.78	2.23	9.3
Pipe P39	538.05	450	130	1276.78	2.23	9.3
Pipe P40	659.49	450	130	1276.78	2.23	9.3
Pipe P41	37.22	150	130	103	1.62	18.53
Pipe P42	53.11	450	130	1168.78	2.04	7.9
Pipe P43	695.73	600	130	-1776.62	1.75	4.22
Pipe P44	248.13	75	130	5	0.31	2
Pipe P45	157.43	600	130	-1786.62	1.76	4.27
Pipe P46	270.87	75	130	5	0.31	2
Pipe P48	186.35	500	130	-1065.7	1.51	3.98
Pipe P49	343.65	600	130	-1796.62	1.77	4.31
Pipe P117	665.43	225	130	122.87	0.86	3.56

Several trials were done to have a stable network of 24 hr EPS in Costain network. The final modelling was in Paida Tank by raising the head by 4.8m and diameter increased to 60. 3.. This (WTP) had 452 nodes and 516 links. 98 % had pressure values within benchmark. After the EPS was performed only 38 % of the nodes had pressure values above 70 m at 7 am and 69 nodes had pressure values above 70 m at 11 am.

Single period analysis of Impresit network was carried out analysis was carried out. Warning messages were generated and ten modification applied is as shown in Tables 4.20 and 4.21. After the modifications, the run gave a good and stable run with pressure, velocity and flow in acceptable manner.as indicated in Table 4.22

Table 4.20: Modification for Impresit WTP SPA

S/N	Pipe	Existing Diameter (mm)	New Diameter(mm)
1.	8	150	300
2.	9	150	300
3.	929	150	300
4.	10	150	400

Table 4.21: Further modification for Impresit network

S/N	Link	Existing Dia(mm)	New Link	New Link Dia(mm)	Setting
1.	P187	225	PRV	225	20
2.	P381	225	PRV	225	30
3.	P418	300	PRV	300	30

Table 4.22: Impresit WTP SPA Node Table

	Elevation (m)	Base Demand (m^3/h)	Demand (m^3/h)	Head (m)	Pressure (m)
Junc J5	221	0	0	344.35	123.35
Junc J7	246	0	0	303.94	57.94
Junc J15	253	3.86	3.86	289.97	36.97
Junc J16	239	3.57	3.57	290.35	51.35
Junc J17	247	3.14	3.14	291.05	44.05
Junc J21	264	0	0	294.15	30.15
Junc J61	265	0.19	0.19	294.15	29.15
Junc J63	264	0	0	294.15	30.15
Junc J70	215	0	0	414.91	199.91
Junc J71	221	0	0	344.35	123.35
Junc J73	236	0	0	344.35	108.35
Junc J74	242	0	0	326.26	84.26
Junc J75	246	0	0	303.94	57.94
Junc J76	262	0	0	303.94	41.94
Junc J77	264	0	0	294.15	30.15
Junc J78	264	0	0	294.15	30.15
Junc J79	265	0.19	0.19	295.79	30.79
Junc J80	282	0.19	0.19	295.8	13.8
Junc J81	244	3.14	3.14	290.87	46.87
Junc J82	259	3.14	3.14	291.32	32.32
Junc J83	237	0	0	290.77	53.77
Junc J84	239	0	0	290.74	51.74
Junc J85	239	1.18	1.18	290.52	51.52
Junc J86	240	1.18	1.18	290.46	50.46
Junc J87	234	3.57	3.57	290.24	56.24
Junc J88	235	3.57	3.57	290.18	55.18
Junc J89	229	3.57	3.57	290.12	61.12
Junc J90	237	3.62	3.62	289.97	52.97
Junc J91	253	3.86	3.86	289.97	36.97
Junc J178	254	0.82	0.82	278.73	24.73

Analysis under peak scenario of Impresit network showed some negative valves at demand points which include junction 342, 343, 344, 345, 346, 355, 356, 810, 818, 819, 823. The modification carried out for the peak scenario of Impresit network to ensure efficient system is shown in Table 4.23. Result of the calibration is depicted in Table 4.24.

Table 4.23: Modification for Peak Scenario of Impresit WTP

S/N	Pipe	Existing Diameter (mm)	New Diameter (mm)
1.	387	100	225
2.	916	100	200
3.	920	100	200
4.	918	100	200
5.	925	100	200

Table 4.24: Impresit WTP Peak Node Table

	Elevation (m)	Base Demand (m^3/h)	Demand (m^3/h)	Head m	Pressure (m)
Junc J5	221	0	0	342.06	121.06
Junc J7	246	0	0	300.36	54.36
Junc J15	253	5.4	5.4	287.94	34.94
Junc J16	239	5	5	288.78	49.78
Junc J17	247	4.4	4.4	290.16	43.16
Junc J21	264	0	0	290.24	26.24
Junc J61	265	2.8	2.8	290.26	25.26
Junc J63	264	0	0	290.24	26.24
Junc J70	215	0	0	414.89	199.89
Junc J71	221	0	0	342.06	121.06
Junc J73	236	0	0	342.06	106.06
Junc J74	242	0	0	323.39	81.39
Junc J75	246	0	0	300.36	54.36
Junc J76	262	0	0	300.36	38.36
Junc J77	264	0	0	290.24	26.24
Junc J78	264	0	0	290.24	26.24
Junc J79	265	2.8	2.8	295.77	30.77
Junc J80	282	2.8	2.8	295.8	13.8
Junc J81	244	4.4	4.4	289.82	45.82
Junc J82	259	4.4	4.4	290.68	31.68
Junc J83	237	0	0	289.62	52.62
Junc J84	239	0	0	289.55	50.55
Junc J85	239	1.6	1.6	289.13	50.13
Junc J86	240	1.6	1.6	288.99	48.99
Junc J87	234	5	5	288.56	54.56
Junc J88	235	5	5	288.44	53.44
Junc J89	229	5	5	288.32	59.31
Junc J90	237	5.1	5.1	287.95	50.95
Junc J91	253	5.4	5.4	287.94	34.94
Junc J92	254	5.4	5.4	287.94	33.94
Junc J179	254	1.1	1.1	278.54	24.54

For extended period simulation in Impresit WTP network: A scenario was created when the head is raised at Bosso Treatment Plant by 15m higher than the elevation of Dutsen Kura Tank. This can be achieved by the use of appropriate pump with Q-H to achieve this. Further modification to obtain a successful run of 24hr extended period stimulus is depicted in Table 4.25

Table 4.25: Adjustment for EPS Impresit Network

S/N	Tank	Diameter (m)	Initial Height (m)	Max Height (m)
1	Dutsen Kura	55	5	7
2	Top Medical	25	1.8	4

This (WTP) had 401 nodes and 466 links. After SPA was performed on the distribution system, only 4 % of the nodes showed positive values of pressure above 70 m which indicated 96 % of the nodes had values of pressure within the benchmark. EPS revealed negative values at different times of the day. At 2 pm 82 % of the nodes showed negative values and at 4pm 89 % of nodes had deficiency in pressure values. After optimization all nodes showed positive values with 74 % of nodes indicated values within the range at 2 pm and 80 % had positive values within the benchmark at 4 pm.

4.3 Result of Leak Detection

Leak detection forms part of the Active Leakage Control (ALC) method. This helps utility in reducing significant amount of water loss in the distribution system.

4.3.1 Identification of leakages using leak detecting equipment

Primary function of leak detecting equipment is to detect the leaks that are not visible to eye or audible to ear. Five major points were identified using the leak detecting equipment. The leaks which in this case are losses are collected and the table below shows the volume of water loss against the time. The imagery showing the coordinates of the sections of the pipeline experimented is shown in Fig 4.1. Leaking point of the Shiroro DMA is shown in Table 4.26

Table 4.26: Leak points and the volume lost

Pipe size in mm	Nodes	Pressure (m)	Coordinate	Lengths between nodes (m)	Leak rate (l/m)	8-hour ($m^{3)}$	Leak at 10^{th} Hour in l/m	Leak at 10^{th} Hour in l/m
150	231	19.96	E231009 N1060366	44.03	.0.4	0.19	0.3	0.28
150	92	20.71	E230968 N1060429	60.30	0.5	0.24	0.5	0.47
100	91	19.72	E231017 N1060507	44.04	0.6	0.29	0.5	0.41
150	238	20.69	E231061 N1060569	729.86	0.5	0.24	0.5	0.43
100	253	16.13	E231261 N1060761	256.78	0.7	0.34	0.4	0.38
			Total volume lost			1.3		

Total length of the pipeline under investigation is 1,135.01m. In 143.57 m length of pipeline, 0.72 m^3 was lost in 8 hours. Total volume lost 1.3 m^3 of treated water DMA. This correspond to 11.16 m^3 per month which is a very huge loss from few leaking points. This has indicated the need for active leakage control because more volume of water was lost unnoticed. The imagery of the leak points is indicated in Figure 4.3.

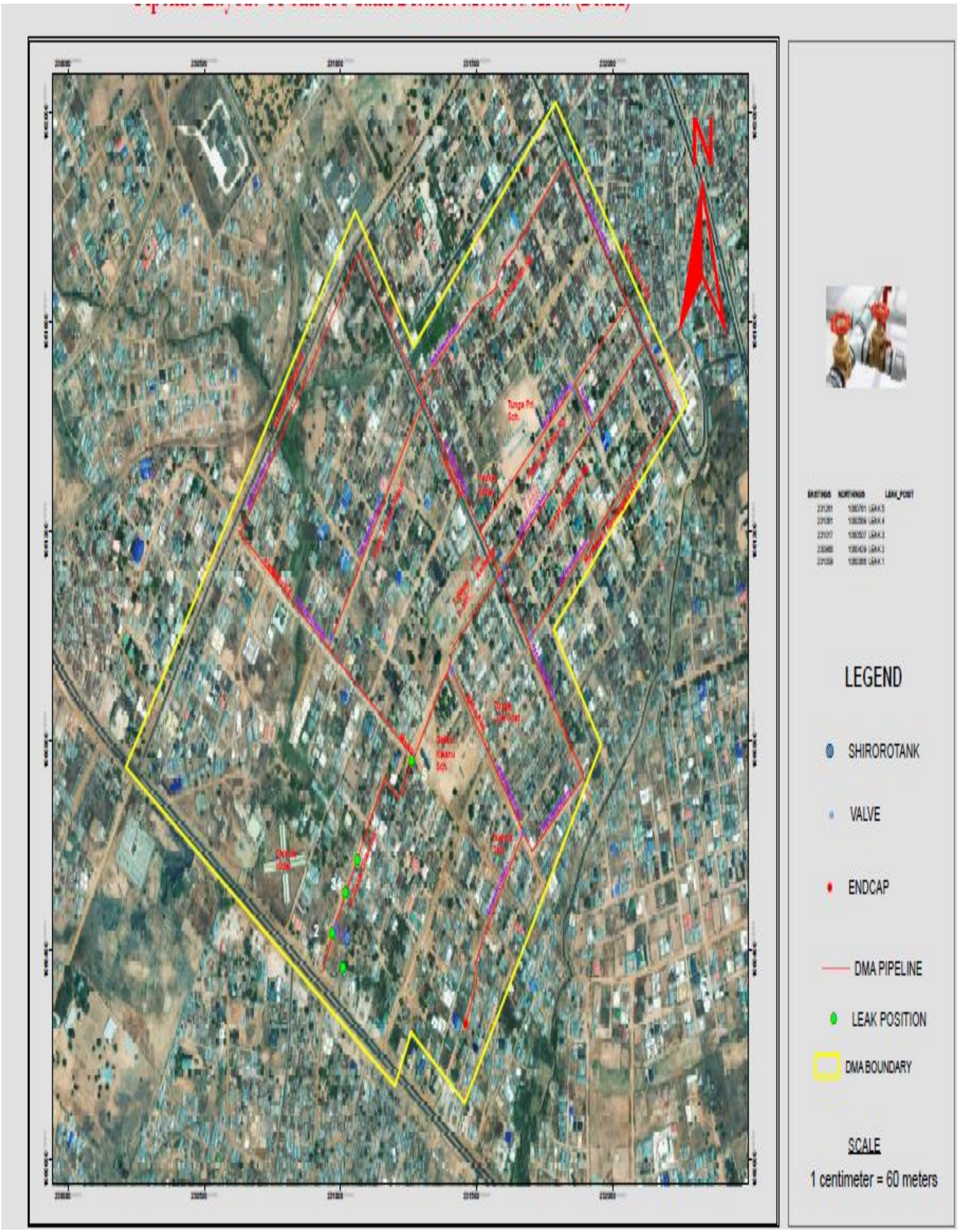


Figure 4.3: Imagery of the leak points

4.4 Result of Pressure and Loss Relationship

Pressure has direct relationship with the leakages in the distribution system. The higher the pressure the more the leaks.

The result of the pressure values and leak rate in the study area is depicted in the Table 4.27. Figure 4.4 depicts the graphical representation of pressure and leak at 3pm of the EPS.

Table 4.27: Pressure and Leak Relationship

Pressure (m)	**Simulaed_Q_{leak} (m^3/h)**
16.08	0.8
24.16	1
13.88	0.7
12.83	0.7
15.32	0.8
13.28	0.7
13.27	0.7
22.06	0.9
10.53	0.6
11.53	0.7
12.79	0.7
12.72	0.7
10.53	0.6
10.55	0.6
13.33	0.7
13.67	0.7
16.72	0.8
18.71	0.9

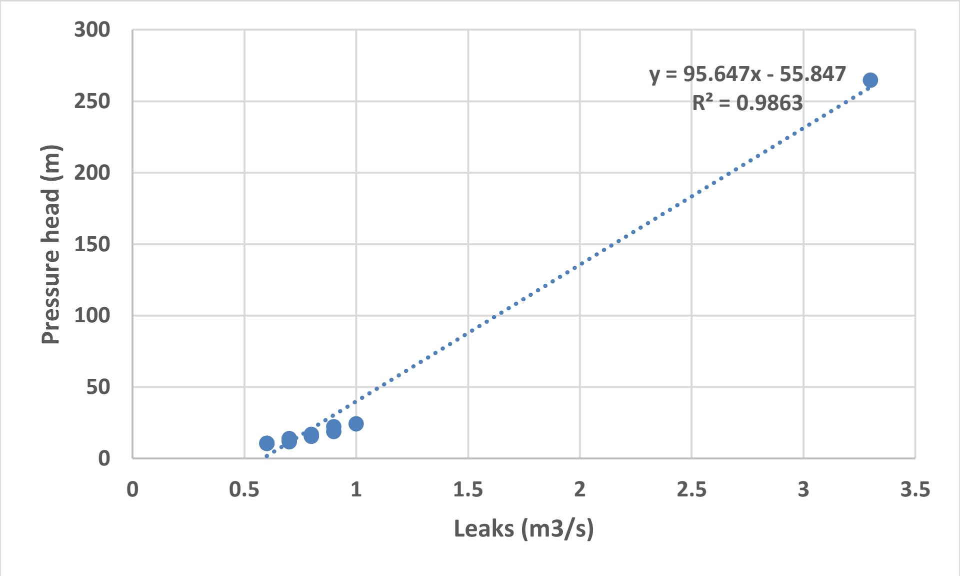


Figure 4.4: Pressure and leak relationship

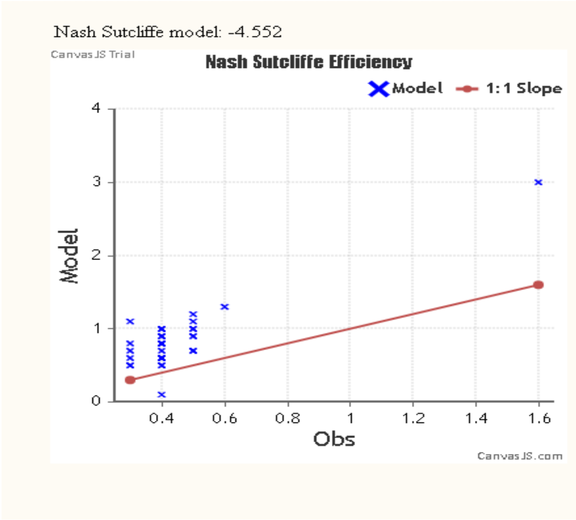
Pearson Product Correlation of pressure and leak was found to be very high positive and statistically significant

4.5 Result for Model Calibration

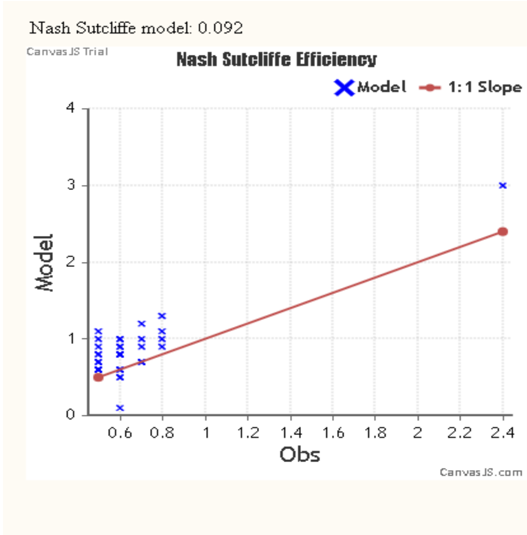
The flow values generated using the emitter and physical measurement of flow on site is depicted in Appendix A.

4.5.1 Modelled and observed data test in NSE

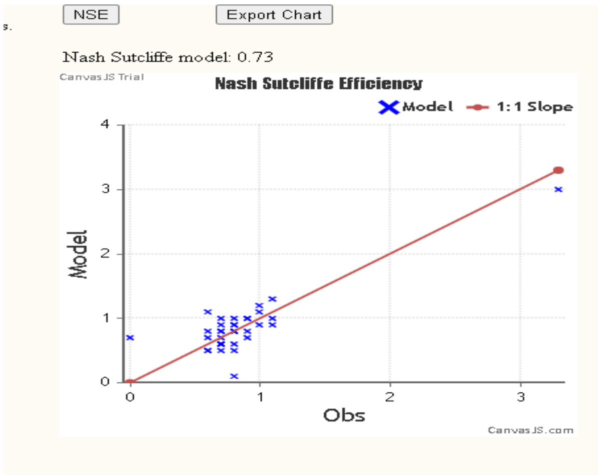
Using the leak coefficients of 0.1, 0.15, 0.2 and 0.3, observed and modelled leakage at 8 hour is presented in Figure 4.5



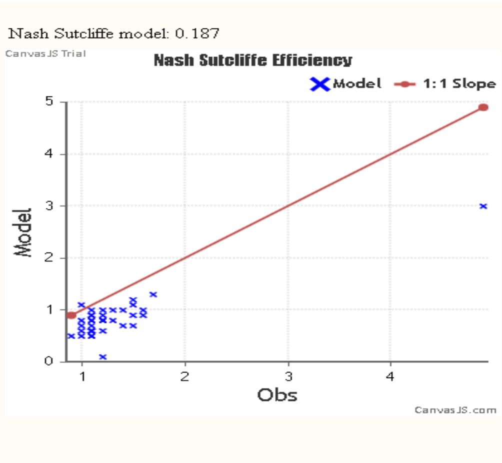
(a) Leak coefficient at 0.1



(b) Leak coefficient at 0.15



(b) Leak coefficient at 0.2

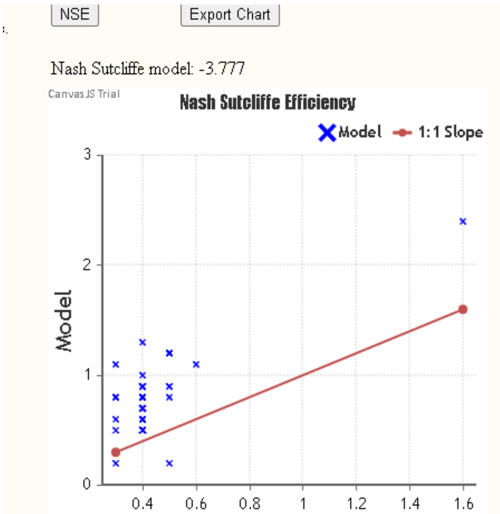


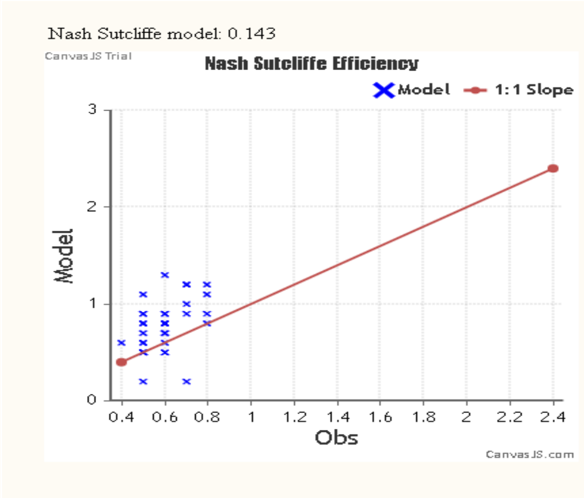
(d) Leak coefficient at 0.3

Figure 4.5: Model Fitting for 8 hour operation

NASH Sutcliffe Efficiency Coefficients at 0.1, 0.15, 0.2 and 0.3 are -4.552, 0.092, 0.73 and 0.187 respectively. These values deviated from the required standards of perfect or nearly perfect match except at 0.2 which gives a nearly perfect match

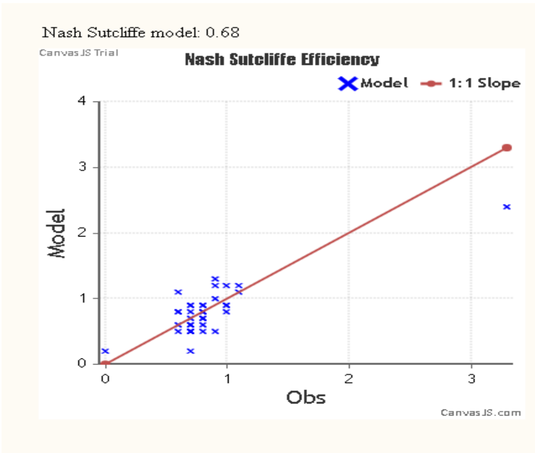
Using the leak coefficients of 0.1, 0.15, 0.2 and 0.3, observed and modelled leakage at 9 hour is presented in Figure 4.6

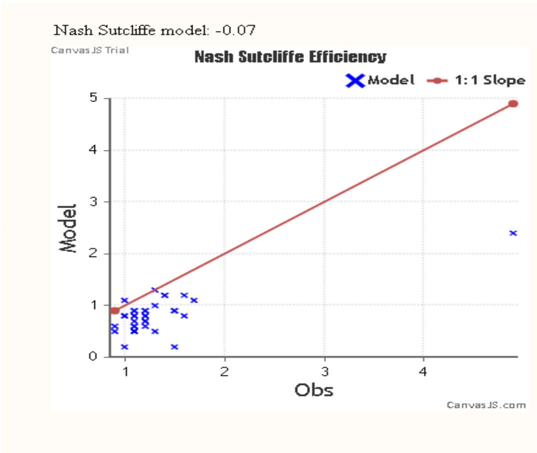




(c) Leak coefficient at 0.1

(b) Leak coefficient at 0.15





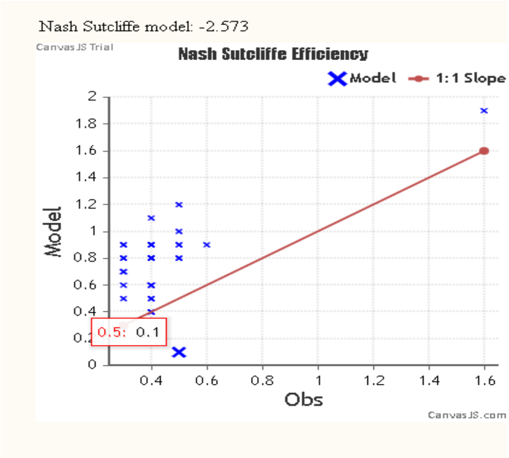
(a) Leak coefficient at 0.2

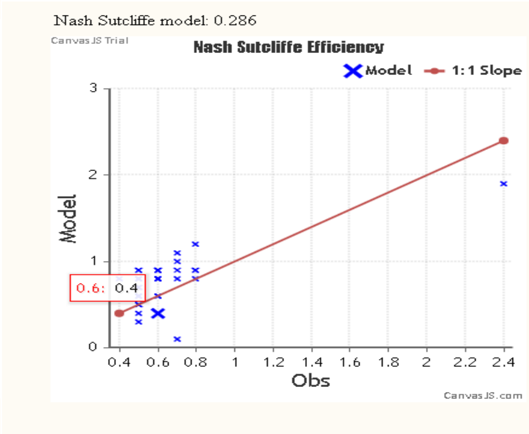
(d) Leak coefficient at 0.3

Figure 4.6: Model Fitting for 9 hour operation

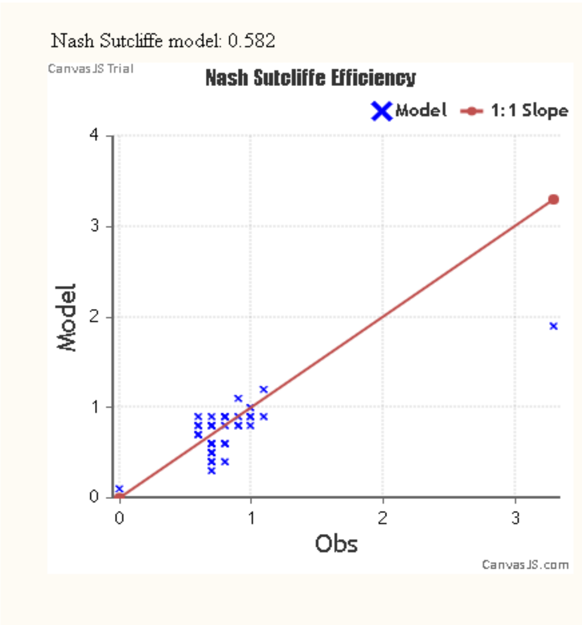
NASH Sutcliffe Efficiency Coefficients at 0.1, 0.15, 0.2 and 0.3 are -3.777, 0.143, 0.68 and -0.07 respectively. These values deviated from the required standards of perfect or nearly perfect match but at 0.2 which gives a near perfect match

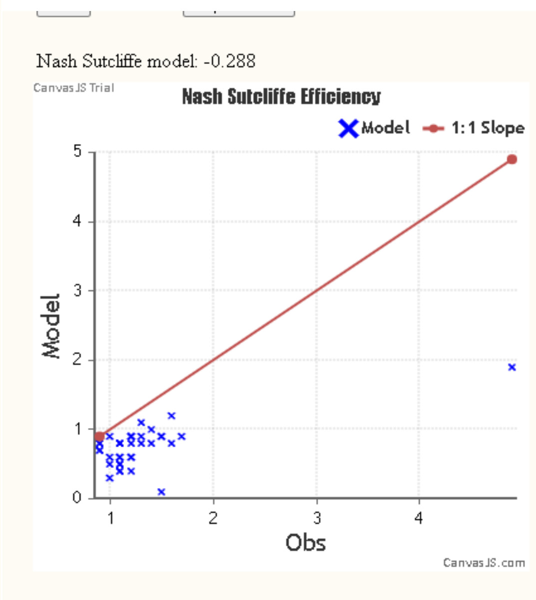
Using the leak coefficients of 0.1, 0.15, 0.2 and 0.3, observed and modelled leakage at 10 hour is presented in Figure 4.7





(d) Leak coefficient at 0.1 (b) Leak coefficient at 0.15



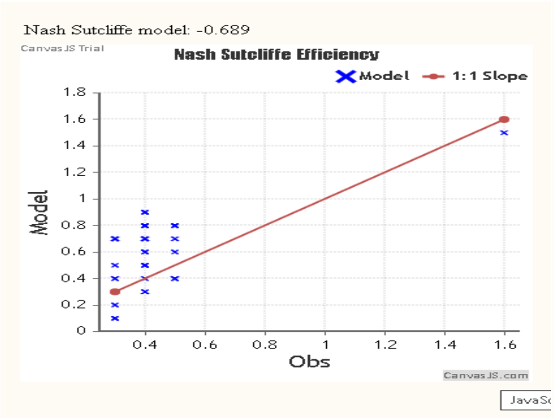


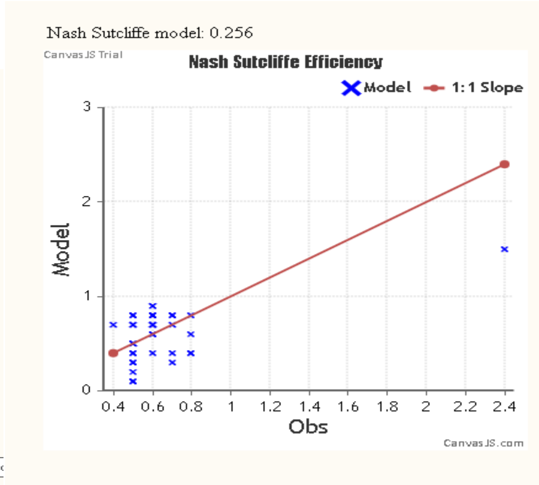
(b) Leak coefficient at 0.1 (d) Leak coefficient at 0.1=

Figure 4.7: Model Fitting for 10 hour operation

NASH Sutcliffe Efficiency Coefficients at 0.1, 0.15, 0.2 and 0.3 are -2.573, 0.286, 0.582 and -0.288 respectively. These values deviated from the required standards of perfect or nearly perfect match except at 0.2 which meets the required standard

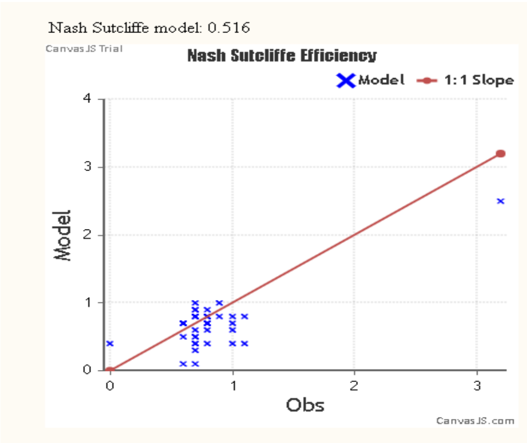
Using the leak coefficients of 0.1, 0.15, 0.2 and 0.3, observed and modelled leakage at 11 hour is presented in Figure 4.8





(e) Leak coefficient at 0.1

(b) Leak coefficient at 0.1



(c) Leak coefficient at 0.1

(d) Leak coefficient at 0.1

Figure 4.8: Model Fitting for 11 hour operation

NASH Sutcliffe Efficiency Coefficients at 0.1, 0.15, 0.2 and 0.3 are -0.689, 0.256, 0.516 and -0.826 respectively. These values deviated from the required standards of perfect or nearly perfect match at except at 0.2 which shows nearly perfect match.

The optimum coefficient is 0.2. Table 4.28 shows the model performance in NSE

Table 4.28: Summary of the Model performance in NSE

	a			
Hour	0.1	0.15	0.2	0.3
8	-4.552	0.092	**0.73**	0.187
9	-3.777	0.143	**0.68**	-0.07
10	-2.573	0.286	**0.582**	-0.288
11	-0.689	0.256	**0.516**	-0.826

Summary of the of the modelled and measured leak is shown in Table 4.29

Table 4.29: Simulated and Observed Leaks at the site

	Base Demand (m^3/h)	Demand (m^3/h)	Pressure (m)	Simu_Q_{leak} (m^3/h)	Obs_Q_{leak} (m^3/h)
249	0.87	1.67	16.08	0.8	1
242	0.87	0.87	24.16	1	0.7
252	3.86	4.61	13.88	0.7	0.8
253	0.87	1.59	12.83	0.7	0.6
250	0.87	1.65	15.32	0.8	0.1
252	3.86	4.59	13.28	0.7	0.9
252	0.87	1.6	13.27	0.7	1
243	0.87	1.81	22.06	0.9	1
254	3.86	4.51	10.53	0.6	0.5
253	0.87	1.55	11.53	0.7	0.6
252	0	0.72	12.79	0.7	0.6
252	0	0.71	12.72	0.7	0.8
254	0.87	1.52	10.53	0.6	1.1
254	3.86	4.51	10.55	0.6	0.8
251	3.86	4.59	13.33	0.7	0.6
251	3.86	4.6	13.67	0.7	0.6
0	3.86	7.11	264.68	3.3	3
248	0.87	1.69	16.72	0.8	0.8
246	0.87	1.74	18.71	0.9	1

Having established this, the values of the model can now be used to predict leakages in the DMA using Artificial neural Network, ANN. The study has shown 17.15% of loss in the network.

4.5.2 Training the network

This study was based on Multi-Layer Perception which was trained and tested using DMA flow data. The objective was to develop an ANN-based model using flow data generated in the selected DMA in Minna, Niger State, Nigeria. The summary of flow logging data is indicated in Table 4.30. The logging and validation values are depicted in Appendix B1.

Table 4.30: Summary of flow logging data

Elevation (m)	Base Demand (m^3/h)	Demand (m^3/h)	Pressure (m)	Simu_Q_{leak} (m^3/h)
248	0.87	1.69	16.72	0.8
246	0.87	1.74	18.71	0.9
246	0.87	1.74	18.71	0.9
252	0.87	1.58	12.71	0.7
251	0.87	1.61	13.62	0.7
248	0.87	1.69	16.61	0.8
250	0.87	1.63	14.55	0.8
249	0.87	1.66	15.52	0.8
255	3.86	4.47	9.19	0.6
247	3.86	4.69	17.14	0.8
254	3.86	4.5	10.29	0.6
248	3.86	4.67	16.29	0.8

In network building several configurations were tried and the one with the "best" prediction efficiency was chosen to be the network training and testing. The weights were adjusted in order to make the actual outputs (predicated) close to the target (measured) outputs of the network.

In this study, pressure, elevation, base demand and demand, were used as the input data and the loss/leakage was the output data.

4.5.3 Testing the network

The next step was to test the performance of the developed ANN model. Data collated from the DMA was used. In order to evaluate the performance of the developed ANN model quantitatively and verify whether there is any underlying trend in performance of ANN model, statistical analysis involving the coefficient of determination (R2), and the root mean square error (RMSE) was computed. RMSE provides information on the short-term performance which is of the variation of predicted values around the measured area. The lower the relative error the more accurate is the estimation. In this model the relative errors errors for training and testing are 0.367 and 0.215 respectively, and R^2 is 0.65.

The result showed that the model built can estimate the amount of leak, given elevation, base demand, demand, pressure and head as variables. This can be useful for water utilities in pipe inspection and maintenance. This study has shown 17.15% of physical or real loss as (NRW). The validation data is given in appendix B2.

The relative errors for samples trained and tested is depicted in Table 4.31. Table 4.32 indicates the model summary in percentages of the valid samples. The hidden layers and their units are depicted in Table 4.33.

Table 4.31: Model Summary

Training sum of square error	Testing sum of square error
0.367	0.215

Table 4.32: Case Processing Summary

Training samples	Testing samples	Validity	% trained	% tested	% valid	Samples excluded
83	28	111	74.8	25.2	100	37

Table 4.33: Network Information

Input layer covariates	Rescaling method for covariates	Number of hidden layer	Number of units in the hidden layer	Activation function	Rescaling method for dependent variable	Error function
Elevation Base Demand Demand Pressure	Standardised	2	3	Hyperbolic function	Standardised	Sum of squares

The predicted and the real values of leaks are depicted in Figure 4.9

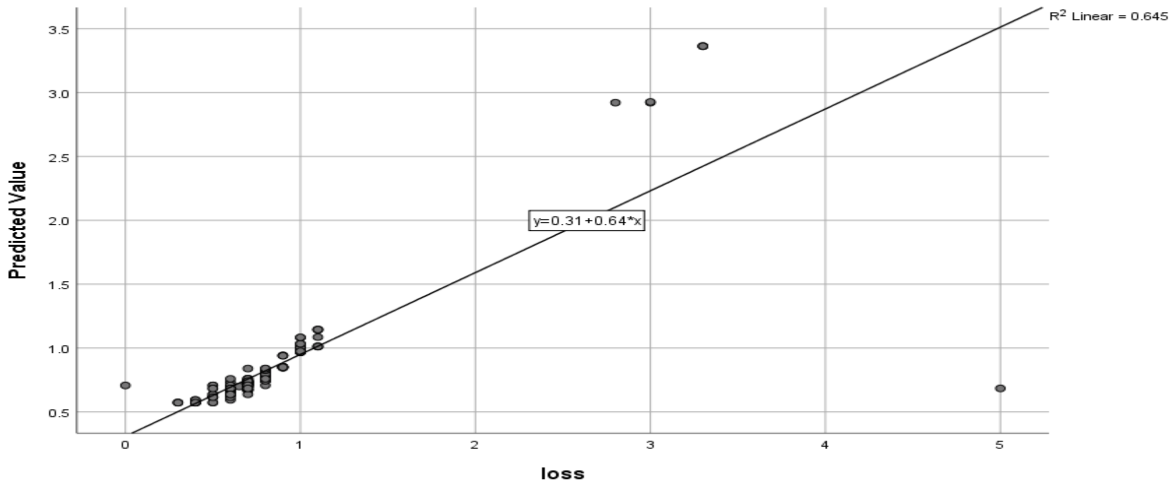


Figure 4.9: Real loss and predicted values of loss

Importance and normalized importance from the model out put is in Table 4.10

Table 4.34: Independent Variable Importance

Variables	Importance	Normalized Importance
elevation	0.289	49.4%
base demand	0.056	9.6%
demand	0.070	11.9%
pressure	0.585	100%

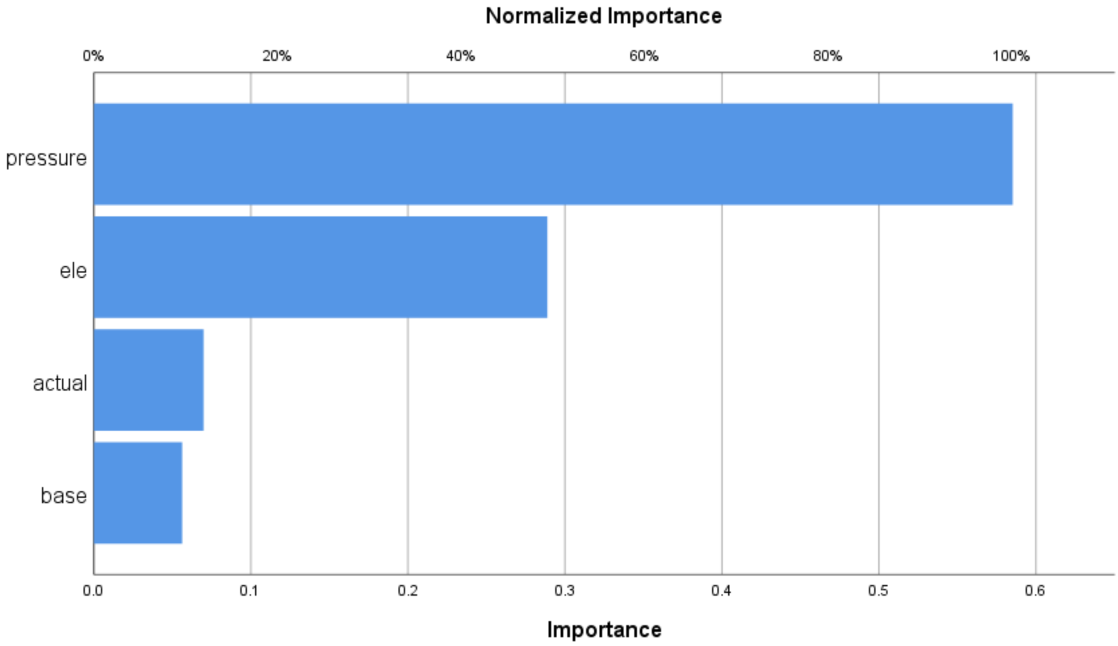


Figure 4.10: Graph showing the importance of the variables used

4.6 Model Validation at M.I. Wushishi Water Distribution Network

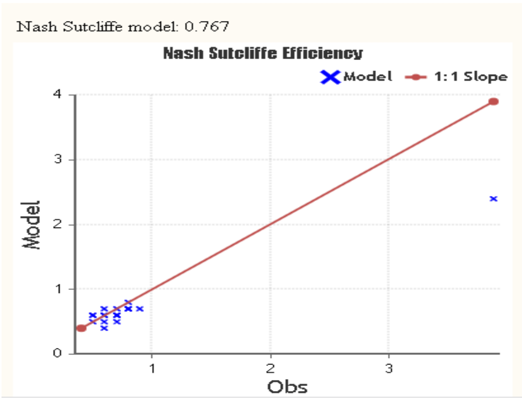
4.6.1 Network information

Network simulation of M. I. Wushishi is depicted in Table 4.35. Discharge Figures 4.24 indicates NASH values at 8 pm, 9 pm and 10 pm of 0.767, 0.668 and 0.601 respectively using Coefficient of 0.2 for validation. The ANN R^2 value for prediction of leaks in M. I. Wushishi is 0.648 as indicated in Figure 4.27. This has suggested that the model has done satisfactorily well.

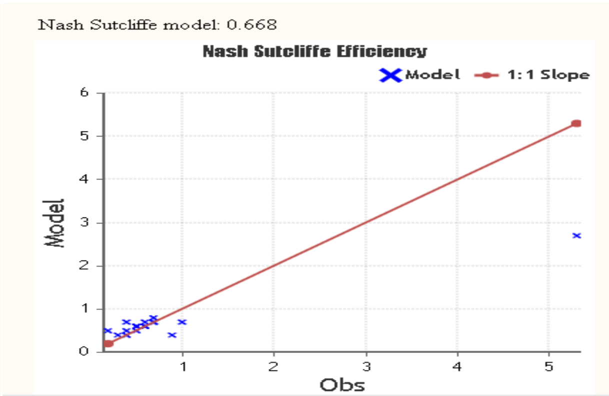
Table 4.35: Model and observed values of leaks in M.I. Wushishi Network

Elevation (m)	Base Demand (m^3/h)	Demand (m^3/h)	Pressure (m)	Simulaed_Q_{leak} (m^3/h)	Observed_Q_{leak} (m^3/h)
249	0.87	1.67	16.08	0.4	0.6
242	0.87	0.87	24.16	0.5	0.7
252	3.86	4.61	13.88	0.6	0.7
253	0.87	1.59	12.83	0.7	0.9
250	0.87	1.65	15.32	0.7	0.8
252	3.86	4.59	13.28	0.7	0.8
252	0.87	1.6	13.27	0.6	0.7
243	0.87	1.81	22.06	0.6	0.5
254	3.86	4.51	10.53	0.4	0.4
253	0.87	1.55	11.53	0.4	0.4
252	0	0.72	12.79	0.5	0.6
252	0	0.71	12.72	0.4	0.4
254	0.87	1.52	10.53	0.5	0.5
254	3.86	4.51	10.55	2.4	3.9
251	3.86	4.59	13.33	0.6	0.6
251	3.86	4.6	13.67	0.6	0.6
0	3.86	7.11	264.68	0.7	0.6
248	0.87	1.69	16.72	0.6	0.7
246	0.87	1.74	18.71	0.6	0.5

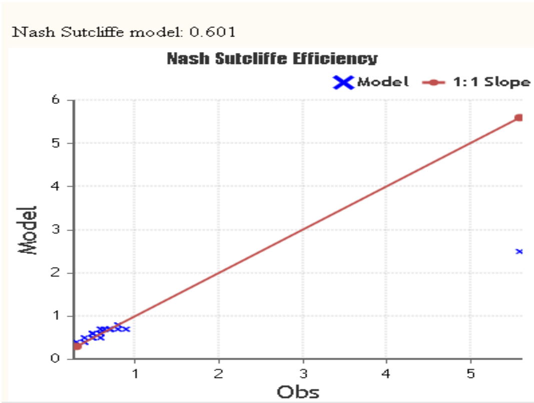
Using the optimum leakage coefficients of 0.2, observed and modelled leakage at 20, 21 and 22 hours is presented in Figure 4.11



(a) Leak coefficient of 0.2 at 20hour



(b) Leak coefficient of 0.2 at 21hour



(b) Leak coefficient of 0.2 at 22hour

Figure 4.11: Validation at 20, 21 and 22 hours water supply

NASH values at optimum leakage coefficient of 0.2 at 20, 21 and 22 hours are 0.767, 0.668 and 0.601 respectively

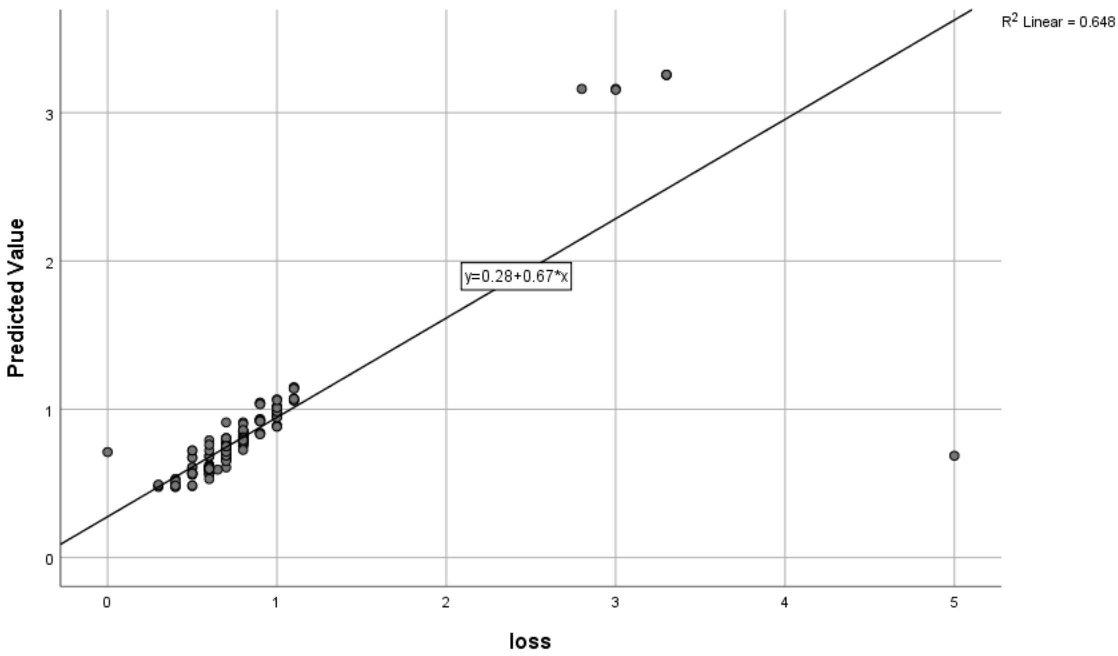


Figure 4.12: Validation of Real loss and predicted values of loss

CHAPTER FIVE

5.0 CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

In the evaluation of the distribution network, the goal was to investigate sustainability in terms of equity in the distribution of pipe-borne water in Minna metropolis. This is believed to provide decision support for efficient pipe-borne water supply in the city. In Minna (WTP), 23 negative values at different joints out of 892 were optimised to make the network efficient. (EPS) revealed 46 % of nodes indicated deficiency in pressure values at 11am. All negative values at the nodes were eliminated after optimisation. In Biwater (WTP) which had 96 nodes and 111 links. 86 % of the nodes showed pressure values within the bench mark of 15 to 15 m. At Costain WTP with 452 nodes and 516 links. 98% had pressure values within benchmark. After the EPS was performed only 171 nodes had pressure values above 70m at 7am and 69 nodes had pressure values above 70m at 11 am. And at Impresit WTP that had 401 nodes and 466 links. After SPA was performed on the distribution system, only 4 % of the nodes showed positive values of pressure above 70m which indicated 96 % of the nodes had values of pressure within the benchmark. EPS revealed negative values at different times of the day. At 2 pm 82 % of the nodes showed negative values and at 4pm 89 % of nodes had deficiency in pressure values. After optimization all nodes showed positive values with 74 % of nodes indicated values within the range at 2pm and 80 % had positive values within the benchmark at 4pm.

Even though, the sections of pipeline in the (DMA) where leak detecting equipment was used were few, the value of loss recorded using the leak detecting equipment is 1.3m^3/h. Total length of the pipeline under investigation is 1,135.01 m. In 143.57 m length of pipeline, 0.72 m^3 was lost in 8 hours. Total volume lost 1.3 m^3 of treated water DMA.

This correspond to 11.16 m^3 per month which is a very huge loss from few leaking points. This has indicated the need for active leakage control because more volume of water was lost unnoticed.

The values of leaks increased as the pressure increased. The leak values decreased as the water level decreased from the reservoir, so also is the pressure level. The highest and the lowest pressure values of 24.16m and 10.53m. Pearson Product Correlation of pressure and leak was found to be very high positive and statistically significant

The (NSE) values obtained for 8th hour, 9th hour, 10th hour and 11th hour are 0.73, 0.68, 0.58 and 0.52 respectively. These values are good indictors of good performance. Model validation for 20 pm, 21 pm and 22 pm hours in M. I. Wushishi Distribution Network gave NASH values of 0.767, 0.668 and 0.601 respectively. NSE values decreased as the head of the reservoir decreased from 8-hour to 11-hour operation. Meaning that the lower the depth of water in the reservoir the less the pressure and by implication the less the leakages at optimum leakage coefficient of 0.2. For validation procedure using ANN, for estimation of leakage at M. I. Wushishi Network, R^2 was 0. 65 and relative errors for training and testing were 0.367 and 0.215 respectively. R^2 of 0.65 is an indication of better performance of the model.in terms of leakage estimation

5.2 Recommendations

i. Leakage exponent of 0.5 was used in the Q_{leak} equation, further studies should consider variation of exponent on the performance of the emitter equation in estimating leakages in Water Distribution Networks

ii. Further studies on application of quantised state approach in Water Distribution Network is recommended.

5.3 Contribution to Knowledge

The research established 63% performance inefficiency of water distribution network in Minna and its environs, which is mostly traced to inappropriate pipe sizing and hydraulic leakage in the system. The Q_{leak} equation used to model the water lost yielded optimum leakage coefficient of 0.2. The relationship between modelled and measured leakages has NASH Sutcliffe Efficiency (NSE) values of 0.73, 0.68, 0.58 and 0.52 at 8-hour, 9-hour, 10-hour and 11-hour operations which are the peak in the distribution system and indicated good performance of the model simulation. The model was validated in the night period with NASH Sutcliffe Efficiency (NSE) values of 0.76, 0.67 and 0.60 for 20-hour, 21-hour and 22-hour operations respectively. At the optimum leakage coefficient of 0.2 water loss through leakage in hydraulic distribution system can be estimated and modelled.

REFERENCES

ADB (2010). Every Drop Counts: Learning from Good Practices in Eight Asian Cities. Asian Development Bank, Manila.

Agunwamba, J. C. (2008). *Water Engineering Systems*. First Edition Immaculate Publications

Alegre, H., Baptista, J. M., Cabrera, E. J., Cubillo, F., Hirner, W., Merkel, W. & Parena, R. (2006). *Performance Indicators for Water Supply Services*, IWA Manual of Best Practice, IWA Publishing

Almasri, M. N. (2012). Optimal design of water distribution networks, An-Najah National University

American Water Works Association (1956). Recommended Practice for Distribution System Records, C900, New York.

Arunkumar, M. & Mariappan, N.V.E. (2011). Water demand analysis of municipal water supply using EPANET software. *International Journal on Applied Bioengineering,* 5(1), 9-18.

AWWA (1992). California Nevada section, Water audit and leak detection guidebook.

AWWA (2003). Committee Report: Applying Worldwide BMPs in water loss control. *Journal AWWA*, 95(8), 65-79.

AWWA (2009). Water Audits and Loss Control Programs: AWWA Manual M36. American Water Works Association, Denver, USA.

Ayanshola, A. M. & Sule, B.F. (2006). Assessment of flow pressure in selected zones of Ilorin township water supply network. *Journal of Research Information in Civil Engineering.* 3(1), 83-101.

Bakirtzis, A. G., Theocharis, J. B., Kiartzis, S. J. & Satsios, K. J. (1995). Short term load forecasting using fuzzy neural networks. *IEEE Transactions on Power Systems,* 10(3), 1518–1524.

Bataineh, S., Al-Anbuky, A. & Al-Aqtash, S. (1996). An expert system for unit commitment and power demand prediction using fuzzy logic and neural networks. *Expert Systems,* 13(1), 29–40.

Baxter, C. W., Zhang, Q., Stanley, S.J., Shariff, R., Tupas, T. & Stark, H. L. (2001). Drinking Water Quality and Treatment: The Use of Artificial Neural Networks. Canadian. *Journal of Civil Engineering,*. 28, 26–35.

Beecher, J. A. (2002). Survey of State Agency Water Loss Reporting Practices. AWWA, Colorado, USA.

Belsito, S., Lombardi, P., Andreussi, P., Banerjee, S.(1998). Leak Detection in Liquified Gas Pipeline by Artificial Neural Network.

Beuken, R. H. S., Lavooij, C. S. W., Bosch, A. & Schaap, P. G. (2006). Low leakage in the Netherlands Confirmed. *Proceedings of the 8th Annual Water Distribution Systems Analysis Symposium* (ASCE), Cincinnati, USA, 1-8

Bhardwaj, V. (2001). Reservoirs, Towers, and Tanks: Drinking Water Storage Facilities, Tech Brief; National Drinking Water Clearinghouse, Tap Magazine.

Biedugnis, S. & Czapczuk, A. (2018). The application of the 'K-nearest neighbour' method to evaluate pressure loss in water supply lines. *Technical Transactions,* 115(1), 141-149, doi: 10.4467/2353737XCT.18. 011.7962.

Boulus, P. F., Karney, B. W. & Lansey, K. E. (2008). Network Components. Comprehensive Water Distribution Systems Analysis Handbook for Engineers and Planners, MHW Soft, Pasadena, CA, 3.23-3.25

Burrows, R., Mulreid, G. & Hayuti, M. (2003). Introduction of a fully dynamic representation of leakage into network modelling studies using EPANET. In: Maksimovic C, Butler D, Memon A eds *Proceedings of the International Conference on Advances in Water Supply Management. Swets & Zeitlinger*, Lisses, 109–118

Bwire, G., Debes A K, Orach C. G, Kagirita A, Ram M, & Komakech H, (2015). Environmental surveillance of Vibrio cholerae O1/O139 in the five African Great Lakes and other major surface water sources in Uganda. Frontier Microbiology, 9: 1560.

Cakmakci, M., Izzet, O., vedal, U., & Lufti, A. (2007). Dimension of Water Loss Through Distribution Systemand Reduction Method in Turkey

Caputo, A. C. & Pelagagge, P. M. (2003). Using neural networks to monitor piping systems, *Process Safety Progress,* 22(2), 119-127.

Carpenter, T., Lambert, A. & McKenzie, R. (2003). Applying the IWA approach to water loss performance indicators in Australia. Water Science and Technology: *Water Supply*, 3(1/2), 153-161.

Chen, X. Y., Chau, K. W. & Busari, A. O (2015) A comparative study of population-based optimization algorithms for downstream river flow forecasting by a hybrid neural network model. *Engineering Application of Artificial Intelligence;* 255-268

Chiipanthenga, M. M. (2008). Investigation of the Suitability of Simple and Rapid Techniques for Leakage Management in Water Distribution Systems: A Case of Blantyre Water Supply Area, Malawi, MSc. Thesis, University of Zimbabwe.

Chipwaila, J. A. (2009). Partitioning of Unaccounted for Water for Zomba City Water Supply System in Malawi. MSc. Thesis, University of Zimbabwe.

City of Harare (2011). Harare Municipality: Water and Sanitation Infrastructure rehabilitation and replacement project. Harare, Zimbabwe.

Cosgrove, W. & Rijsberman, F. (2000). World Water Vision: Making Water Everybody Business 1st ed. London: Earthscan Publications.

Dawidowicz, J. (2012). Expert System for Evaluation of Water Distribution System Created with an Inductive Inference. *Rocznik Ochrona Srodowiska*, 14; 650-659.

Dawidowicz, J. (2017). Evaluation of a pressure head and pressure zones in water distribution systems by artificial neural networks. *Neural Computin & Application*, doi:10.1007/s00521-017- 2844-8.

De Silva, D., Mashford, J. & Burn, S. (2011). *Computer Aided Leak Location and Sizing in Pipe Networks*, Urban Water Security Research Alliance Technical Publication No. 17 (2nd Edition).

Dongquan, Z. O., Hao-zheng, W. G., Ji-ning, C. N. & Hao-chang, W. G. (2009). Parameters uncertainty analysis of urban rainfall-runoff simulation, *Shuikexue Jinzhan/Advances in Water Science*, 20(1), 45-51.

Dongwoo., J & Gyewoon, C. (2017). Estimation of Non-Revenue Water Ratio Using MRA and ANN in Water Distribution Networks

Fanner, P. (2004). *Assessing Real Losses, including Component Analysis and Economic Considerations: A Practical Approach*. Water21 Article No 6, IWA Publishing, London.

Fantozzi, M. & Fantozzi, S. (2008). Practical Application of Economic Intervention in Active Leakage Control. Paper to Water Loss Conference, Feb. 14-15, Waitakere City, Auckland.

Farley, M. & Trow, S. (2003). *Losses in water distribution networks*. A Practitioner's Guide to Assessment, Monitoring and Control, IWA Publishing, London.

Farley, M. (2003). Non-revenue water - International Best Practice for Assessment, Monitoring and Control Annual CWWA Water, Wastewater and Solid Waste Conference. Atlantis, Paradise Island, Bahamas.

Farley, M., Wyeth, G., Ghazali, Z., Istandar, A. & Singh, S. (2008). *The Manager's Non-Revenue Water Handbook*: A Guide to Understanding Water Losses, Washington DC, USA.

Federal Survey, (1976), Topographical and Map of Minna. Sheet 42. Edition 2. Produced and published by Federal Surveys, Nigeria.

Feng, J, & Zhang, H. (2006). A New Fault Detection and Diagnosis Method for Oil Pipeline based on Rough Set and Neural Network. In: Liu, D., Fei, S., Hou, Z., Zhang, H., Sun, C. (eds) Advances in Neural Networks

Germanopoulous, G. (1985) “A Technical Note on the Inclusion of Pressure Dependent Demand and Leakage Terms in Water Supply Network Models,” *Civil Engineering and Environmental Systems*, 2(3), 171-179.

Gianfredi, M., Luigi, B, Daniele L., Riccardo M., Antonietta, S. & Orazio, G. (2016) A methodology to estimate leakages in water distribution networks based on inlet flow data analysis. *ScienceDirect, Procedia Engineering*, 162; 411-418

Gumbo, B. & Van der Zaag, P. (2002). Water Losses and the Political constraints to Demand Management: The case of the City of Mutare, Zimbabwe. *Physics and Chemistry of the Earth, Parts A/B/C,* 27(11-22), 805-813.

Hadzilacos, T., Kalles, D., Preston, N., Melbourne, P., Camarinopoulos L., Eimermacher, M., Kallidromitis, V. Frondistou-Yannas, S. & Saegrov, S. (2002) A Water Mains Rehabilitation Decision Support System. *Computers, Environment and Urban Systems,* 24; 215-232.

Hamideh, F., Muhammadreza, J., Babak, A. & Jafar, Y. (2021). Ghazizadeh Leakage Detection in Water Distribution Networks Using Hybrid Feedforward Artificial Neural Networks. *AQUA - Water Infrastructure, Ecosystems and Society,* 70 (5): 637–653.

Haykin, S. (1994). *Neural Networks: A Comprehensive Foundation*; Macmillan: New York, NY, USA,

Hunaidi, O. (2010). Leakage Management for Water Distribution Infrastructure – Report 1: Results of DMA Experiments in Regina, SK. Canada, National Research Council.

IRC, (1996). Construction Innovation: defecting corrosion in water mains, 1(3), 13-22.

IWA (International Water Association) (2003) Water Loss Task Force

Izabela, R. & Jan, S. (2019). Detection and Localization of Water Leaks in Water

Izquierdo, J., López, P.A. & Martínez, R.P. (2007). Fault detection in water supply systems using hybrid (theory and data-driven) modelling. *Mathematical and Computer Modelling,* 46; 341-350.

Jacek, D., Andrzej, C. & Jacek, P. (2018). The Application of Artificial Neural Network in the Assessment of Pressure Losses in Water Pipes in the Design of Water Distribution System. *ResearchGate, Volume/Tom,* 20; *292-308*

Jain, A., Varshney, A. K. & Joshi, U. C. (2001). Short-Term Water Demand Forecast Modelling at IIT Kanpur Using Artificial Neural Networks. *Water Resour. Manag.* 15, 299–321.

Kanakoudis, V. & Tsitsifli, S.. (2010). Results of an urban water distribution network performance evaluation attempt in Greece. *Urban Water Journal,* 7(5), 267-285.

Karadirek, I. E., Kara, S., Yilmaz, G., Muhammedtoglu, A., Muhammedtoglu, G., (2012). Implementation of Hydraulic Modelling for Water-Loss Reduction Through Pressure Management.

Karthikeyan, L., Kumar, D. N., Graillot, D. & Gaur, S. (2013). Prediction of ground water levels in the uplands of a tropical coastal riparian wetland using artificial neural networks. *Water resources management,* 27(3), 871-883.

Kingdom, B., Liemberger, R. & Marin, P. (2006). The Challenge of Reducing Non-Revenue Water (NRW) in Developing Countries, Paper No. 8 available on websitehttp://siteresources.worldbank.org/INTWSS/Resources/WSS8fin4.pdf

Lambert , A.(2001). Review of performance indicators for real losses from water supply systems. *AQUA*, 3(2), 56-62.

Lambert, A. & Lalonde, A. (2005). Using practical predictions of Economic Intervention Frequency to calculate Short-run Economic Leakage Level, with or without Pressure Management. Proceedings of IWA Specialised Conference "Leakage 2005", Halifax, Nova Scotia, Canada

Lambert, A. O. & Mckenzie, R. S. (2002). A Water losses management and techniques. Water Science and Technology: *Water Supply*, 2(4), 1–20,

Lambert, A. O., Brown, T.G., Takizawa, M. & Weimer, D. A. (1999). Review of Performance Indicators for Real Losses from Water Supply Systems, *AQUA*, 48(6), 227–237.

Liemberger, R. & Farley, M. (2005). Developing a non -revenue water reduction strategy part I: Investigation and assessing water losses.

Liemberger, R.. Brothers, K., Lambert, A., McKenzie, R., Rizzo, R. & Waldron, T. (2002). Water Loss Performance Indicators. *Water Supply*, 2(4), 1–20

Maind, S. B. & Wankar, P. (2014). Research Paper on Basic of Artificial Neural Network. *International Journal on Recent and Innovation Trends in Computing and Communication*, 2(1), 96 – 100

Makaya, E. & Hensel, O. (2014). The contribution of leakage water to total water loss in Harare, Zimbabwe. *International Researchers*, 3(3), 14-23.

Makaya, E. & Hensel, O. (2015). Modelling flow dynamics in water distribution networks using Artificial Neural Networks - A leakage detection technique. *International Journal of Engineering, Science and Technology*, 7; 1-15

Makaya, E. (2014). Water distribution systems losses. A guide for operation and maintenance. Cibew Publishing Company, Botswana, ISBN 978-99968-0-269-0.

Mamlook, R. & Al-Jayyous, H. (2003) Fuzzy sets analysis for leak detection in infrastructure systems: a proposed methodology. *Cleaning Techniques and Environmental Policy*, 6(1), 26-33.

Marin, P. (2009). Public Private Partnerships for Urban Water Utilities: A Review of Experiences in Developing Countries. Trends and Policy Options 8, Public-Private Infrastructure Advisory Facility, Washington DC, World Bank.

Marques, R. C. & Monteiro, A. J. (2003). Application of Performance Indicators in Water Utilities Management, a case Study in Portugal. *Water Science and Technology*, 44 (2-3), 95-102

Marunga, A., Hoko, Z. & Kaseke, E. (2006). Pressure management as a leakage reduction and water demand management tool: The case of the City of Mutare, Zimbabwe. *Physics and Chemistry of the Earth*, 31(15-16), 763-770.

Mathis, M., Kunkel, G. & Howley, A. C. (2008). Water Loss Audit Manual for Texas Utilities. Texas Water Development Board, Report 367, March 2008.

McIntosh, A. C. (2003). *Asian Water Supplies: Reaching the Urban Poor*, Asian Development Bank and IWA Publishing.

Mckenzie, R. S. (1999). SANFLOW User Guide: South Africa Water Research Commission, WRC Report No. TT 109/99 Pretoria, South Africa, Water Research Commission.

McKenzie, R. S. (2002). ECONOLEAK User Guide. South Africa Water Research Commission, WRC Report TT 169/02, Pretoria, South Africa.

McKenzie, R. S. & Bhagwan, J. N. (2004). Leakage Management - Introduction to WRC Tools to Manage Non-revenue Water. Available from: http://www.wrc.org.za /downloads/special%2 0publications/leakage_managment.pdf, Accessed 21 March 2012.

Mitchell, V., Mein, R. G. & Mcmahon, T. (2000). Modelling the Urban Water Cycle. *Environmental Modelling & Software*, 16; 615-629.

Mounce, S.R. & Machell, J. (2006). Burst Detection Using Hydraulic Data from Water Distribution Systems with Artificial Neural Networks. *Urban Water, Journal,* 3; 21–31.

Msiza, I. S., Nelwamondo, F. V. & Marwala, T. (2008). Water Demand Prediction Using Artificial Neural Networks and Support Vector Regression. *Journal. Comput,* 3; 1–8.

Mulwafu, C. G., Chavula, A., Ferguson, B. G. & Nkhoma, G. C. (2003). *Water demand management in Malawi: Problems and prospects for its promotion*. Physics and Chemistry of the Earth, Parts A/B/C, 28; 601-612.

Mwendera, E. J, Hazelton, N. D., Robinson, P., Tjijenda, K. & Chavula, G. (2003) Overcoming constraints to the implementation of water demand management in southern Africa. Physics and Chemistry of the Earth, Parts A/B/C, 28; 761-778.

Niger State WASH Policy, (2020). Niger State Water Sanitation and Hygiene Policy. Ministry of Water Resources and Dams Development

NISWASEC (Niger State Water and Sewage Corporation) (2019) Monitoring and Evaluation Record

OFWAT (2010). Service and Wales and delivery-performance of the water companies in England 2009-10 report, OFWAT, UK.

Ostadi, H. & Azimi, F. (2015). Comparing the ability of ANN artificial neural networks and ARIMA method for predicting steel price. *Journal of Scientific Research and Development*, 2(3), 5-10

Park, H. J. (2006). A Study to develop strategies for Proactive Water Loss Management. PhD Thesis, Georgia State University, USA.

Radivojević, D., Milićević, D. & Petrović, N. (2007). *Technical Performance Indicators*, IWA Best Practice for Water Mains and the First Steps in Serbia

Raman, H. & Chandramouli, V. (1996). Deriving a General Operating Policy for Reservoirs using Neural Network. *Journal of Water Resources. Planning. Manag*, 122; 342–347.

Rizzo, A., Pearson, D., Stephenson, M. & Harper, N. (2004). Apparent water loss control: A practical approach. *IWA Water Loss Task Force Water*, 6(3), 21-39.

Rossman, L. A. (2000). EPANET 2 User Manual. Water Supply and Water Resources Division National Risk Management Research Laboratory, Cincinnati, OH, EPA/600/R-00/057.

Rothert, S. (2000). "Water conservation and demand management potential in southern Africa: an untapped river". *International Journal Water*, 1(1), 118–144.

Seago, C., Bhagwan, J. & McKenzie, R. (2004). Benchmarking leakage from water reticulation systems in South Africa. *Journal of Water SA*, 30(5), 25-32.

Segmasin Nig. Ltd. (1998) Assessment of unaccounted water in Bida and New Bussa, Nigeria. A report prepared in collaboration with Parkman Nig Ltd and submitted to Niger State

Shamseldin, A. Y. (1997). Application of neural network technique to rainfall-runoff modeling. *Journal of Hydrology*, 199; 272-294.

Shinozuka, M. & Liang, J.(2005), On-line damage identification of water delivery systems. *Proceedings of Engng Mech Conf*.

Simbeye, I. (2010). Managing Non-Revenue Water. NRW-Sourcebook for Trainers. WAVE Pool. InWEnt.

Soldevila, A. Blesa, J. Fernandez-Canti, R. M.,Tornil-Sin, S. & Puigi, V. (2019). Data Driven Approach for Leak Localization in Water Distribution Networks Using Pressure Sensors and Spatial Interpolation. *Water (Basel)*, 11 (7), 1500-1513

Sturm, R. & Thornton, J., (2007). Water Loss Control in N.A.; More Cost Effective than Customer Side Conservation – Why Wouldn't You Do It? Paper presented to the CA/NVAWWA Spring Conference, Las Vegas. Government.

Tate, E. L. (2000). A water balance study of the upper White Nile basin flows in the late nineteenth century. *Hydrological Sciences Journal*, 46(2), 301-318.

The Central Public Health and Environmental Engineering, New Delhi, India (1999) Manual of Water Supply and Treatment. New Delhi (India): Expert Committee constituted by the Government of India, Central Public Health and Environmental Engineering Organisation (CPHEEO), Indian Ministry of Urban Development.

The Mathworks Inc., (2002). Neural Network Toolbox; for use with MATLAB, User's guide Version 4.

Thornton, J. (2003). Managing Leakage by Managing Pressure. Water 21, October 2003

Thornton, J. & Lambert, A. O. (2007). Pressure management extends infrastructure life and reduces unnecessary energy costs. Proceedings of IWA International Specialised Conference 'Water Loss 2007' held in Bucharest, Cyprus, ISBN 978-973-7681-24-9, 1-9.

Thornton, J. & Lambert, A.O. (2005). Progress in practical prediction of pressure: leakage, pressure: burst frequency and pressure: consumption relationships. Paper to IWA Special Conference "Leakage 2005", September 12-14, Halifax, Canada.

Turki, A. Mohd, S. Y. Norfaizal, Y., Shamsuddin S. K. & Ariffin, A. (2020) A Review on Water Leakage Detection Method in the Water Distribution Network. *Journal of Advanced Research in Fluid Mechanics and Thermal Sciences;* 152-163.

Twort, A. C.; Law F. M., Crowley F. W., Ratnayaka D. D. (1994). Water supply. Edition 4. 1994

USAID/RTI E-WASH Pipeline Network Analysis, Niger State (2020) *WASH Assessment Report*

UKWIR, (2005). Leakage Index Curve and the Longer-Term Effects of Pressure Management; Towards Best Practice for the Assessment of Supply Pipe Leakage.

USAID & WBI (2010). The Manager's Non-Revenue Water Handbook for Africa: A guide to understanding Water Losses, Washington DC, Development Alternatives Inc (DAI).

Vela, A., Perez, R. & Espert, V. (1991). Incorporation of leakages in the mathematical model for a water distribution network. *Proceedings of the 2nd International Conference on Computer Methods in Water Resources,* Marrakesh, Morocco, 245-257.

Walling, D. E., (2003). "Recent trends in the suspended sediment loads of the world's rivers". *Journal of Global and Planetary change,* 39: 111-126,

Wallingford, H. R. (2003). Handbook for assessment of catchment of water demand and use. Water pipeline design guidelines. "*The Environmental Management and Protection*".

Walski, T. M., Chase, D. V., Savic, D. A., Grayman, W., Beckwith, S. & Koelle, E. (2008). Advanced Water Distribution Modelling and Management. Haestad Methods

Warren, R. P. (2005). The service pipe – a forgotten asset in leak detection. *Paper to IWA Special Conference* "Leakage 2005", September 12-14, Halifax, Canada.

Washington State University Uniform Design Standards (1998). Water Distribution Systems, Part C, Section 02660-4.

WHO, (2000) World Health Organization, Global Water Supply and Sanitation Assessment. *Global Situation Report*

WHO, (2001) World Health Organization, Global Water Supply and Sanitation Assessment. *Global Situation Report*

Wolday, B. (2005). Comprehensive asset management in water supply and sanitation.

World Bank, (2006). A Demand Driven Approach in Service Delivery: The Community Water and Sanitation Project, Ghana http://siteresources.worldbank.org/INTWSS/Resources/ghana.pdf. Accessed on 13/04/2012.

WSP, (2009). Water Operators Partnerships: African Utility Performance Assessment. Water and Sanitation Program (WSP) - Africa, The World Bank, Nairobi, Kenya.

Wu, Z. Y., Sage, P. & Turtle, D. (2011). Pressure-Dependent Leak Detection Model and its application to a District Water System. *Water Resources Plan Management*, 136; 116-128

WWAP, (2014). United Nations World Water Assessment Programme, United Nations, New York.

Yitbarek, A. M. (2018). Population Forecasting for Design of Water Supply System in Injibara Town, Amhara Region, Ethiopia. Civil and Environmental Research www.iiste.org ISSN 2224-5790 (Paper) ISSN 2225-0514 (Online) 10(10)

Zealand, C. M., Burn, D. H. & Simonovic, S. P. (1999). Short Term Streamflow Forecasting using Artificial Neural Networks. *Journal of Hydrology*, 214, 32-48.

Zhang, G., Patuwo, B. E. & Hu, M. Y. (1998). Forecasting with artificial neural networks: The state of the art. *International Journal of Forecasting*, 14, 35–62

Zhang, J., Walter, G. G., Miao, Y., Wayne, W. N. (1995). Wavelet neural networks for function learning. *IEEE Transactions on Signal Processing*, 43(6), 1485–1497

Zhang, Y. W. (2004). Fault detection and diagnosis of nonlinear processes using improved kernel independent component analysis (KICA) and Support Vector Machine (SVM). *Industrial & Engineering Chemistry Research*, 47(18), 6961-6971.

Appendix A: Logging Data for DMA analysis

Elevation (m)	Base Demand (m^3/h)	Demand (m^3/h)	Pressure (m)
249	0.87	1.67	16.08
242	0.87	0.87	24.16
252	3.86	4.61	13.88
253	0.87	1.59	12.83
250	0.87	1.65	15.32
252	3.86	4.59	13.28
252	0.87	1.6	13.27
243	0.87	1.81	22.06
254	3.86	4.51	10.53
253	0.87	1.55	11.53
252	0	0.72	12.79
252	0	0.71	12.72
254	0.87	1.52	10.53
254	3.86	4.51	10.55
251	3.86	4.59	13.33
251	3.86	4.6	13.67
0	3.86	7.11	264.68
248	0.87	1.69	16.72
246	0.87	1.74	18.71
246	0.87	1.74	18.71
252	0.87	1.58	12.71
251	0.87	1.61	13.62
248	0.87	1.69	16.61
250	0.87	1.63	14.55
249	0.87	1.66	15.52
255	3.86	4.47	9.19
247	3.86	4.69	17.14
254	3.86	4.5	10.29
248	3.86	4.67	16.29
239	3.86	4.88	26.11
237	3.86	4.92	27.84
240	3.86	4.85	24.73
241	3.86	4.83	23.46
241	3.86	4.96	30.04
235	3.86	4.97	31.01
247	3.86	4.76	20.37
250	0.87	1.63	14.54
249	0.87	1.67	15.8
242	0.87	0.87	23.88
252	3.86	4.6	13.6
253	0.87	1.58	12.55
250	0.87	1.65	15.04

Elevation (m)	Base Demand (m^3/h)	Demand (m^3/h)	Pressure (m)
252	3.86	4.58	13
252	0.87	1.59	12.99
243	0.87	1.8	21.79
254	3.86	4.5	10.25
253	0.87	1.54	11.25
252	0	0.71	12.52
252	0	0.71	12.44
254	0.87	1.51	10.26
254	3.86	4.5	10.27
251	3.86	4.58	13.06
251	3.86	4.59	13.4
0	3.86	7.11	264.41
248	0.87	1.68	16.44
246	0.87	1.73	18.44
246	0.87	1.73	18.44
252	0.87	1.58	12.43
251	0.87	1.6	13.35
248	0.87	1.68	16.34
250	0.87	1.63	14.28
249	0.87	1.65	15.25
255	3.86	4.46	8.91
247	3.86	4.68	16.87
254	3.86	4.49	10.02
248	3.86	4.66	16.02
239	3.86	4.88	25.83
237	3.86	4.91	27.56
240	3.86	4.85	24.45
241	3.86	4.82	23.18
241	3.86	4.95	29.74
235	3.86	4.97	30.73
247	3.86	4.76	20.08
250	0.87	1.63	14.27
249	0.87	1.66	15.53
242	0.87	0.87	23.6
252	3.86	4.59	13.32
253	0.87	1.57	12.27
250	0.87	1.64	14.77
252	3.86	4.57	12.73
252	0.87	1.58	12.72
243	0.87	1.8	21.51
254	3.86	4.49	9.98
253	0.87	1.53	10.98
252	0	0.7	12.24

Elevation (m)	Base Demand (m^3/h)	Demand (m^3/h)	Pressure (m)
252	0	0.7	12.17
254	0.87	1.5	9.98
254	3.86	4.49	10
251	3.86	4.58	12.79
251	3.86	4.58	13.12
0	3.86	7.11	264.14
248	0.87	1.67	16.17
246	0.87	1.72	18.17
246	0.87	1.72	18.16
252	0.87	1.57	12.16
251	0.87	1.59	13.08
248	0.87	1.67	16.06
250	0.87	1.62	14
249	0.87	1.64	14.98
255	3.86	4.45	8.64
247	3.86	4.67	16.6
254	3.86	4.48	9.75
248	3.86	4.65	15.75
239	3.86	4.87	25.55
237	3.86	4.9	27.29
240	3.86	4.84	24.18
241	3.86	4.82	22.91
241	3.86	4.95	29.44
235	3.86	4.96	30.44
247	3.86	4.75	19.8
250	0.87	1.62	13.99

Appendix B1 Modelled and Observed leak values

	Elevation (m)	Base Demand (m^3/h)	Demand (m^3/h)	Pressure (m)	Simu_Q_{leak} (m^3/h)	Measured_site_Q_{leak} (m^3/h)
Junc J251	249	0.87	1.67	16.08	0.8	1
Junc J252	242	0.87	0.87	24.16	1.0	0.7
Junc J253	252	3.86	4.61	13.88	0.7	0.8
Junc J254	253	0.87	1.59	12.83	0.7	0.6
Junc J255	250	0.87	1.65	15.32	0.8	0.1
Junc J256	252	3.86	4.59	13.28	0.7	0.9
Junc J257	252	0.87	1.6	13.27	0.7	1
Junc J258	243	0.87	1.81	22.06	0.9	1
Junc J259	254	3.86	4.51	10.53	0.6	0.5
Junc J260	253	0.87	1.55	11.53	0.7	0.6
Junc J261	252	0	0.72	12.79	0.7	0.6
Junc J262	252	0	0.71	12.72	0.7	0.8
Junc J263	254	0.87	1.52	10.53	0.6	1.1
Junc J264	254	3.86	4.51	10.55	0.6	0.8
Junc J266	251	3.86	4.59	13.33	0.7	0.6
Junc J267	251	3.86	4.6	13.67	0.7	0.6
Junc J268	0	3.86	7.11	264.68	3.3	3
Junc J269	248	0.87	1.69	16.72	0.8	0.8
Junc J270	246	0.87	1.74	18.71	0.9	1
Junc J271	246	0.87	1.74	18.71	0.9	0.8
Junc J272	252	0.87	1.58	12.71	0.7	0.7
Junc J273	251	0.87	1.61	13.62	0.7	0.5
Junc J274	248	0.87	1.69	16.61	0.8	0.9
Junc J275	250	0.87	1.63	14.55	0.8	0.9
Junc J278	249	0.87	1.66	15.52	0.8	0.6
Junc J279	255	3.86	4.47	9.19	0.6	0.5
Junc J280	247	3.86	4.69	17.14	0.8	0.8
Junc J281	254	3.86	4.5	10.29	0.6	0.7
Junc J282	248	3.86	4.67	16.29	0.8	0.8
Junc J283	239	3.86	4.88	26.11	1.0	1.1
Junc J284	237	3.86	4.92	27.84	1.1	1
Junc J288	240	3.86	4.85	24.73	1.0	0.9
Junc J304	241	3.86	4.83	23.46	1.0	1.2
Junc J305	241	3.86	4.96	30.04	1.1	0.9
Junc J311	235	3.86	4.97	31.01	1.1	1.3
Junc J314	247	3.86	4.76	20.37	0.9	0.7
Junc J895	250	0.87	1.63	14.54	0.8	0.5
Junc J251	249	0.87	1.67	15.8	0.8	0.9
Junc J252	242	0.87	0.87	23.88	1.0	0.2
Junc J253	252	3.86	4.6	13.6	0.7	0.8
Junc J254	253	0.87	1.58	12.55	0.7	0.5
Junc J255	250	0.87	1.65	15.04	0.8	0.7
Junc J256	252	3.86	4.58	13	0.7	0.9
Junc J257	252	0.87	1.59	12.99	0.7	0.5

	Elevation (m)	Base Demand (m^3/h)	Demand (m^3/h)	Pressure (m)	Simu_Q_{leak} (m^3/h)	Measured_site_Q_{leak} (m^3/h)
Junc J258	243	0.87	1.8	21.79	0.9	1.2
Junc J259	254	3.86	4.5	10.25	0.6	0.8
Junc J260	253	0.87	1.54	11.25	0.7	0.2
Junc J261	252	0	0.71	12.52	0.7	0.5
Junc J262	252	0	0.71	12.44	0.7	0.9
Junc J263	254	0.87	1.51	10.26	0.6	0.8
Junc J264	254	3.86	4.5	10.27	0.6	1.1
Junc J266	251	3.86	4.58	13.06	0.7	0.6
Junc J267	251	3.86	4.59	13.4	0.7	0.6
Junc J268	0	3.86	7.11	264.41	3.3	2.4
Junc J269	248	0.87	1.68	16.44	0.8	0.6
Junc J270	246	0.87	1.73	18.44	0.9	1.3
Junc J271	246	0.87	1.73	18.44	0.9	0.5
Junc J272	252	0.87	1.58	12.43	0.7	0.7
Junc J273	251	0.87	1.6	13.35	0.7	0.5
Junc J274	248	0.87	1.68	16.34	0.8	0.9
Junc J275	250	0.87	1.63	14.28	0.8	0.8
Junc J278	249	0.87	1.65	15.25	0.8	0.7
Junc J279	255	3.86	4.46	8.91	0.6	0.6
Junc J280	247	3.86	4.68	16.87	0.8	0.8
Junc J281	254	3.86	4.49	10.02	0.6	0.5
Junc J282	248	3.86	4.66	16.02	0.8	0.8
Junc J283	239	3.86	4.88	25.83	1.0	0.9
Junc J284	237	3.86	4.91	27.56	1.0	0.8
Junc J288	240	3.86	4.85	24.45	1.0	0.9
Junc J304	241	3.86	4.82	23.18	1.0	1.2
Junc J305	241	3.86	4.95	29.74	1.1	1.2
Junc J311	235	3.86	4.97	30.73	1.1	1.1
Junc J314	247	3.86	4.76	20.08	0.9	1
Junc J895	250	0.87	1.63	14.27	0.8	0.5
Junc J251	249	0.87	1.66	15.53	0.8	0.8
Junc J252	242	0.87	0.87	23.6	1.0	0.1
Junc J253	252	3.86	4.59	13.32	0.7	0.8
Junc J254	253	0.87	1.57	12.27	0.7	0.4
Junc J255	250	0.87	1.64	14.77	0.8	0.9
Junc J256	252	3.86	4.57	12.73	0.7	0.8
Junc J257	252	0.87	1.58	12.72	0.7	0.6
Junc J258	243	0.87	1.8	21.51	0.9	0.8
Junc J259	254	3.86	4.49	9.98	0.6	0.8
Junc J260	253	0.87	1.53	10.98	0.7	0.9
Junc J261	252	0	0.7	12.24	0.7	0.3
Junc J262	252	0	0.7	12.17	0.7	0.6
Junc J263	254	0.87	1.5	9.98	0.6	0.7
Junc J264	254	3.86	4.49	10	0.6	0.9
Junc J266	251	3.86	4.58	12.79	0.7	0.6

	Elevation (m)	Base Demand (m^3/h)	Demand (m^3/h)	Pressure (m)	Simu_Q_{leak} (m^3/h)	Measured_site_Q_{leak} (m^3/h)
Junc J267	251	3.86	4.58	13.12	0.7	0.5
Junc J268	0	3.86	7.11	264.14	3.3	1.9
Junc J269	248	0.87	1.67	16.17	0.8	0.4
Junc J270	246	0.87	1.72	18.17	0.9	0.9
Junc J271	246	0.87	1.72	18.16	0.9	0.8
Junc J272	252	0.87	1.57	12.16	0.7	0.5
Junc J273	251	0.87	1.59	13.08	0.7	0.5
Junc J274	248	0.87	1.67	16.06	0.8	0.9
Junc J275	250	0.87	1.62	14	0.7	0.4
Junc J278	249	0.87	1.64	14.98	0.8	0.6
Junc J279	255	3.86	4.45	8.64	0.6	0.8
Junc J280	247	3.86	4.67	16.6	0.8	0.6
Junc J281	254	3.86	4.48	9.75	0.6	0.7
Junc J282	248	3.86	4.65	15.75	0.8	0.9
Junc J283	239	3.86	4.87	25.55	1.0	0.9
Junc J284	237	3.86	4.9	27.29	1.0	0.8
Junc J288	240	3.86	4.84	24.18	1.0	0.9
Junc J304	241	3.86	4.82	22.91	1.0	1
Junc J305	241	3.86	4.95	29.44	1.1	1.2
Junc J311	235	3.86	4.96	30.44	1.1	0.9
Junc J314	247	3.86	4.75	19.8	0.9	1.1
Junc J895	250	0.87	1.62	13.99	0.7	0.8
Junc J251	249	0.87	1.65	15.25	0.8	0.7
Junc J252	242	0.87	0.87	23.32	1.0	0.4
Junc J253	252	3.86	4.58	13.04	0.7	0.8
Junc J254	253	0.87	1.56	11.99	0.7	0.3
Junc J255	250	0.87	1.63	14.49	0.8	0.7
Junc J256	252	3.86	4.57	12.45	0.7	0.8
Junc J257	252	0.87	1.58	12.44	0.7	0.5
Junc J258	243	0.87	1.79	21.24	0.9	0.8
Junc J259	254	3.86	4.48	9.71	0.6	0.7
Junc J260	253	0.87	1.52	10.71	0.7	0.1
Junc J261	252	0	0.69	11.97	0.7	0.2
Junc J262	252	0	0.69	11.9	0.7	0.4
Junc J263	254	0.87	1.49	9.71	0.6	0.7
Junc J264	254	3.86	4.48	9.73	0.6	0.5
Junc J266	251	3.86	4.57	12.51	0.7	0.5
Junc J267	251	3.86	4.58	12.85	0.7	0.3
Junc J268	0	3.86	7.11	263.86	3.2	1.5
Junc J269	248	0.87	1.67	15.89	0.8	0.8
Junc J270	246	0.87	1.72	17.89	0.8	0.9
Junc J271	246	0.87	1.72	17.89	0.8	0.7
Junc J272	252	0.87	1.56	11.89	0.7	0.4
Junc J273	251	0.87	1.59	12.8	0.7	0.5
Junc J274	248	0.87	1.66	15.79	0.8	0.8

	Elevation (m)	Base Demand (m^3/h)	Demand (m^3/h)	Pressure (m)	Simu_Q_{leak} (m^3/h)	Measured_site_Q_{leak} (m^3/h)
Junc J275	250	0.87	1.61	13.73	0.7	0.6
Junc J278	249	0.87	1.64	14.7	0.8	0.6
Junc J279	255	3.86	4.44	8.37	0.6	0.7
Junc J280	247	3.86	4.67	16.33	0.8	0.8
Junc J281	254	3.86	4.48	9.48	0.6	0.1
Junc J282	248	3.86	4.65	15.47	0.8	0.4
Junc J283	239	3.86	4.87	25.28	1.0	0.4
Junc J284	237	3.86	4.9	27.01	1.0	0.6
Junc J288	240	3.86	4.84	23.91	1.0	0.7
Junc J304	241	3.86	4.81	22.64	1.0	0.8
Junc J305	241	3.86	4.94	29.14	1.1	0.4
Junc J311	235	3.86	4.96	30.16	1.1	0.8
Junc J314	247	3.86	4.74	19.51	0.9	0.3
Junc J895	250	0.87	1.61	13.72	0.7	0.9

Appendix B2: Model Validation Result in ANN

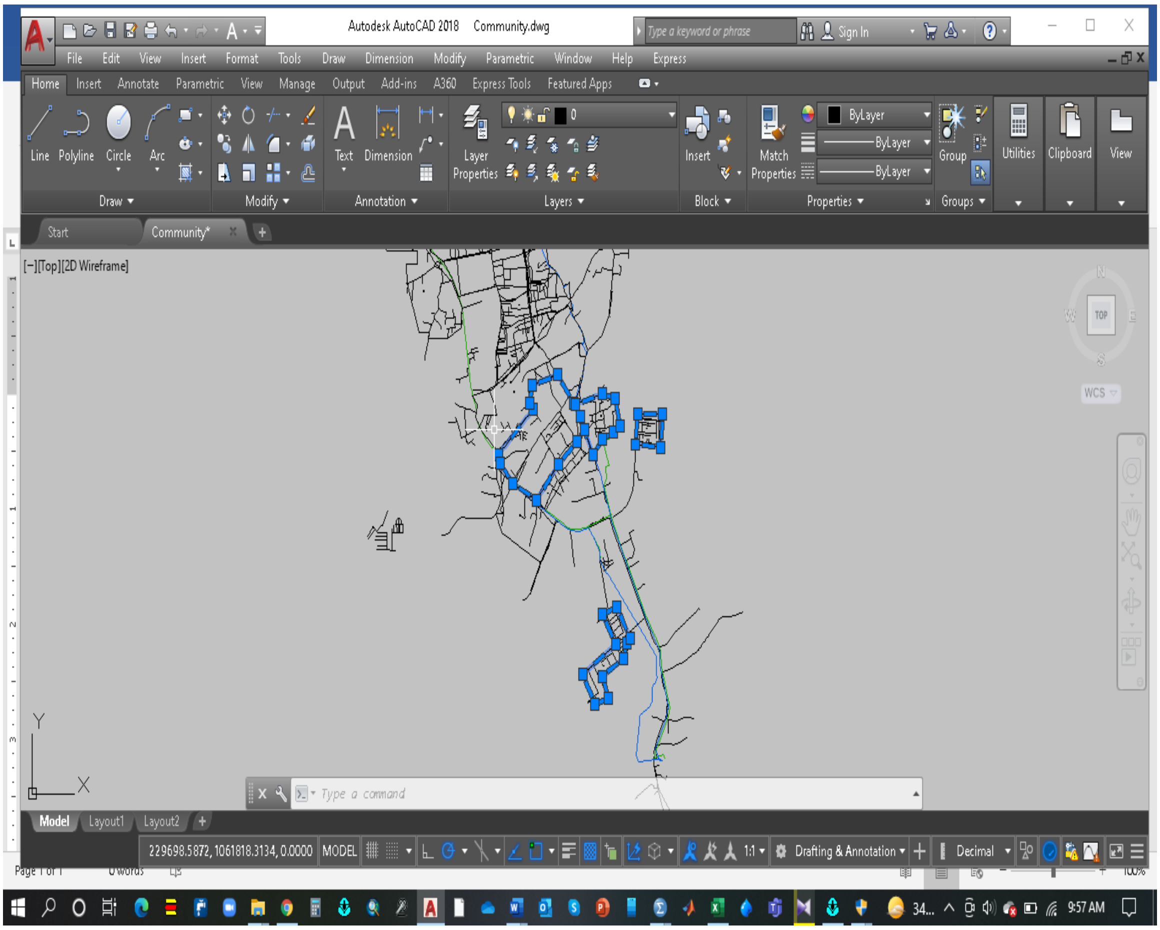
Elevation (m)	Base demand (m^3/h)	Actual Demand (m^3/h)	Pressure (m)	Leak (m^3/h)	MLP_PredictedValue (m^3/h)
249	0.87	1.67	16.08	0.8	0.79
242	0.87	0.87	24.16	1	0.98
252	3.86	4.61	13.88	0.7	0.7
253	0.87	1.59	12.83	0.7	0.7
250	0.87	1.65	15.32	0.8	0.77
252	3.86	4.59	13.28	0.7	0.69
252	0.87	1.6	13.27	0.7	0.72
243	0.87	1.81	22.06	0.9	0.95
254	3.86	4.51	10.53	0.6	0.63
253	0.87	1.55	11.53	0.7	0.68
252	0	0.72	12.79	0.7	0.72
252	0	0.71	12.72	0.7	0.72
254	0.87	1.52	10.53	0.6	0.65
254	3.86	4.51	10.55	0.6	0.63
251	3.86	4.59	13.33	0.7	0.7
251	3.86	4.6	13.67	0.7	0.71
0	3.86	7.11	264.68	3.3	3.29
248	0.87	1.69	16.72	0.8	0.81
246	0.87	1.74	18.71	0.9	0.87
246	0.87	1.74	18.71	0.9	0.87
252	0.87	1.58	12.71	0.7	0.71
251	0.87	1.61	13.62	0.7	0.73
248	0.87	1.69	16.61	0.8	0.81
250	0.87	1.63	14.55	0.8	0.76
249	0.87	1.66	15.52	0.8	0.78
255	3.86	4.47	9.19	0.6	0.6
247	3.86	4.69	17.14	0.8	0.8
254	3.86	4.5	10.29	0.6	0.62
248	3.86	4.67	16.29	0.8	0.78
239	3.86	4.88	26.11	1	1.03
237	3.86	4.92	27.84	1.1	1.08
240	3.86	4.85	24.73	1	1
241	3.86	4.83	23.46	1	0.97
241	3.86	4.96	30.04	1.1	1.07
235	3.86	4.97	31.01	1.1	1.15
247	3.86	4.76	20.37	0.9	0.86
250	0.87	1.63	14.54	0.8	0.76
249	0.87	1.67	15.8	0.8	0.79
242	0.87	0.87	23.88	1	0.98
252	3.86	4.6	13.6	0.7	0.7
253	0.87	1.58	12.55	0.7	0.69
250	0.87	1.65	15.04	0.8	0.76

Elevation (m)	Base demand (m^3/h)	Actual Demand (m^3/h)	Pressure (m)	Leak (m^3/h)	MLP_PredictedValue (m^3/h)
252	3.86	4.58	13	0.7	0.69
252	0.87	1.59	12.99	0.7	0.71
243	0.87	1.8	21.79	0.9	0.95
254	3.86	4.5	10.25	0.6	0.62
253	0.87	1.54	11.25	0.7	0.67
252	0	0.71	12.52	0.7	0.72
252	0	0.71	12.44	0.7	0.72
254	0.87	1.51	10.26	0.6	0.65
254	3.86	4.5	10.27	0.6	0.62
251	3.86	4.58	13.06	0.7	0.7
251	3.86	4.59	13.4	0.7	0.7
0	3.86	7.11	264.41	3.3	3.29
248	0.87	1.68	16.44	0.8	0.81
246	0.87	1.73	18.44	0.9	0.86
246	0.87	1.73	18.44	0.9	0.86
252	0.87	1.58	12.43	0.7	0.7
251	0.87	1.6	13.35	0.7	0.73
248	0.87	1.68	16.34	0.8	0.81
250	0.87	1.63	14.28	0.8	0.75
249	0.87	1.65	15.25	0.8	0.78
255	3.86	4.46	8.91	0.6	0.59
247	3.86	4.68	16.87	0.8	0.8
254	3.86	4.49	10.02	0.6	0.62
248	3.86	4.66	16.02	0.8	0.77
239	3.86	4.88	25.83	1	1.02
237	3.86	4.91	27.56	1	1.07
240	3.86	4.85	24.45	1	0.99
241	3.86	4.82	23.18	1	0.96
241	3.86	4.95	29.74	1.1	1.07
235	3.86	4.97	30.73	1.1	1.14
247	3.86	4.76	20.08	0.9	0.85
250	0.87	1.63	14.27	0.8	0.75
249	0.87	1.66	15.53	0.8	0.78
242	0.87	0.87	23.6	1	0.97
252	3.86	4.59	13.32	0.7	0.69
253	0.87	1.57	12.27	0.7	0.69
250	0.87	1.64	14.77	0.8	0.76
252	3.86	4.57	12.73	0.7	0.68
252	0.87	1.58	12.72	0.7	0.71
243	0.87	1.8	21.51	0.9	0.94
254	3.86	4.49	9.98	0.6	0.62
253	0.87	1.53	10.98	0.7	0.67
252	0	0.7	12.24	0.7	0.71

Elevation (m)	Base demand (m^3/h)	Actual Demand (m^3/h)	Pressure (m)	Leak (m^3/h)	MLP_PredictedValue (m^3/h)
252	0	0.7	12.17	0.7	0.71
254	0.87	1.5	9.98	0.6	0.64
254	3.86	4.49	10	0.6	0.62
251	3.86	4.58	12.79	0.7	0.69
251	3.86	4.58	13.12	0.7	0.7
0	3.86	7.11	264.14	3.3	3.29
248	0.87	1.67	16.17	0.8	0.8
246	0.87	1.72	18.17	0.9	0.86
246	0.87	1.72	18.16	0.9	0.86
252	0.87	1.57	12.16	0.7	0.7
251	0.87	1.59	13.08	0.7	0.72
248	0.87	1.67	16.06	0.8	0.8
250	0.87	1.62	14	0.7	0.75
249	0.87	1.64	14.98	0.8	0.77
255	3.86	4.45	8.64	0.6	0.59
247	3.86	4.67	16.6	0.8	0.79
254	3.86	4.48	9.75	0.6	0.61
248	3.86	4.65	15.75	0.8	0.77
239	3.86	4.87	25.55	1	1.02
237	3.86	4.9	27.29	1	1.07
240	3.86	4.84	24.18	1	0.99
241	3.86	4.82	22.91	1	0.96
241	3.86	4.95	29.44	1.1	1.06
235	3.86	4.96	30.44	1.1	1.14
247	3.86	4.75	19.8	0.9	0.85
250	0.87	1.62	13.99	0.7	0.75
249	0.87	1.65	15.25		0.78
242	0.87	0.87	23.32		0.97
252	3.86	4.58	13.04		0.69
253	0.87	1.56	11.99		0.68
250	0.87	1.63	14.49		0.76
252	3.86	4.57	12.45		0.68
252	0.87	1.58	12.44		0.7
243	0.87	1.79	21.24		0.94
254	3.86	4.48	9.71		0.61
253	0.87	1.52	10.71		0.66
252	0	0.69	11.97		0.71
252	0	0.69	11.9		0.71
254	0.87	1.49	9.71		0.64
254	3.86	4.48	9.73		0.61
251	3.86	4.57	12.51		0.69
251	3.86	4.58	12.85		0.69
0	3.86	7.11	263.86		3.29

Elevation (m)	Base demand (m^3/h)	Actual Demand (m^3/h)	Pressure (m)	Leak (m^3/h)	MLP_PredictedValue (m^3/h)
248	0.87	1.67	15.89		0.8
246	0.87	1.72	17.89		0.85
246	0.87	1.72	17.89		0.85
252	0.87	1.56	11.89		0.69
251	0.87	1.59	12.8		0.72
248	0.87	1.66	15.79		0.8
250	0.87	1.61	13.73		0.74
249	0.87	1.64	14.7		0.77
255	3.86	4.44	8.37		0.58
247	3.86	4.67	16.33		0.79
254	3.86	4.48	9.48		0.61
248	3.86	4.65	15.47		0.77
239	3.86	4.87	25.28		1.02
237	3.86	4.9	27.01		1.06
240	3.86	4.84	23.91		0.98
241	3.86	4.81	22.64		0.95
241	3.86	4.94	29.14		1.06
235	3.86	4.96	30.16		1.14
247	3.86	4.74	19.51		0.84
250	0.87	1.61	13.72		0.74

Appendix D: Network Loops



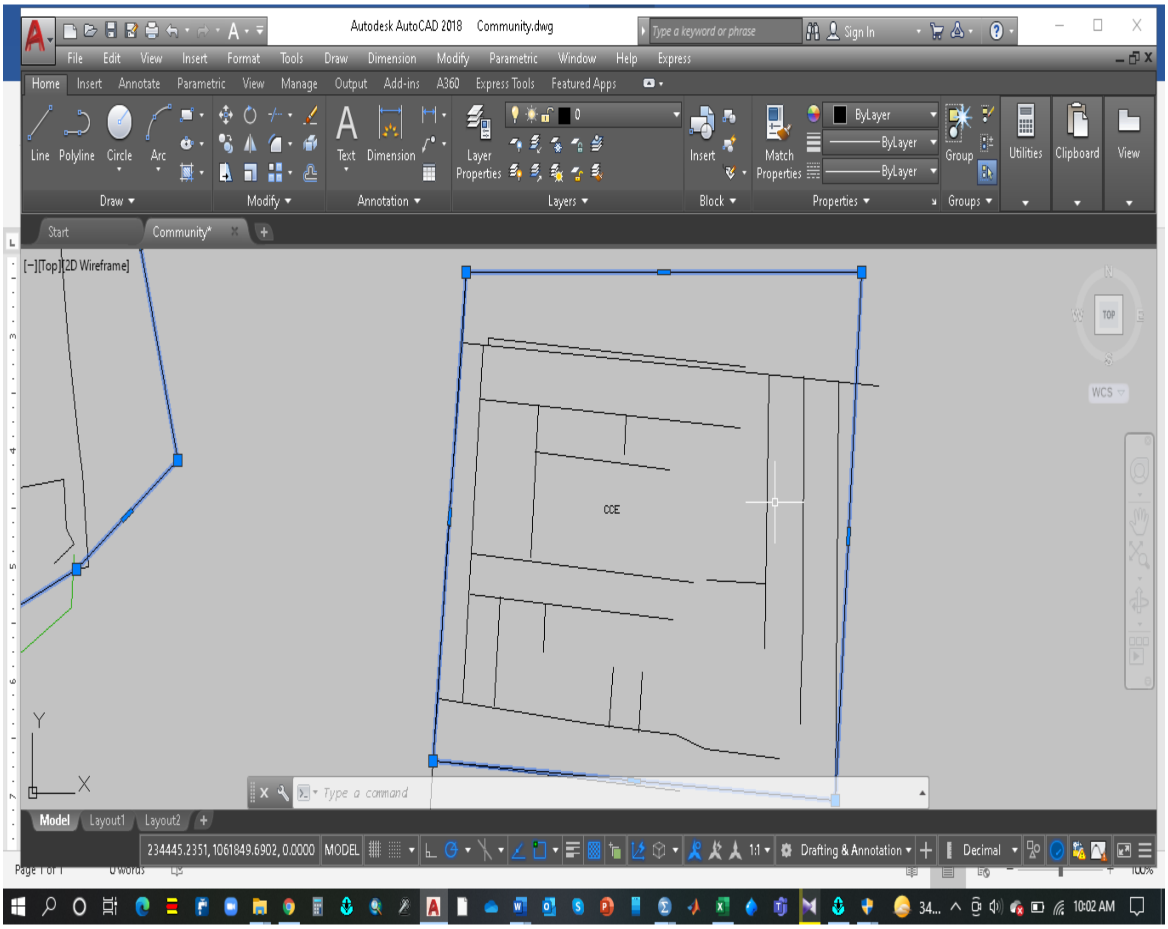


Plate II: Community CCE in AutoCAD

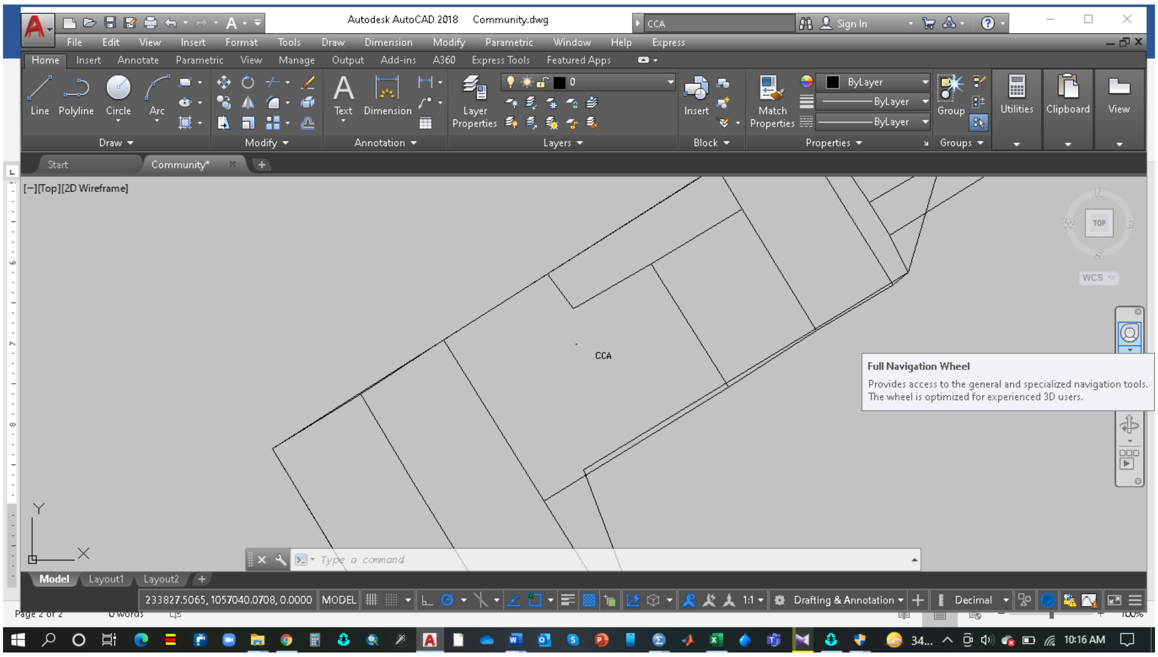


Plate III: Community CCA in AutoCAD

Appendix E Field Work on Leak Detection and Measurement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