# OPTIMIZATION OF RESERVOIR PRESSURE FOR MANAGEMENT OF WATER LOSSES IN DISTRIBUTION NETWORK: CASE OF KIMILILI WATER SUPPLY SCHEME, KENYA

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A thesis submitted in partial fulfillment of the requirements of the award of degree of Master of Science in Water Resources Engineering of Masinde Muliro University of Science and Technology

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## DECLARATION

This thesis is my original work prepared with no other than the indicated sources and support and has not been presented elsewhere for a degree or any other award.

**Celsus Murenjekha Shilehwa**

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## CERTIFICATION

The undersigned certify that they have read and hereby recommend for acceptance of Masinde Muliro University of Science and Technology a thesis entitled; **"Optimization of Reservoir Pressure for Management of Water Losses in Distribution Network: Case of Kimilili Water Supply Scheme, Kenya"**.

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## DEDICATION

This work is dedicated to my parents Mr and Mrs Shilehwa for their support, my wife Nelius Waiyego Kariuki for her sincere love, understanding and support.

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## ABSTRACT

Potable water supply distribution networks are designed and constructed to convey treated water from the water treatment plant to end users. In developing countries, urban water supplies are mostly intermittent, typically ranging from 2 hours to 4 hours in a day and experiences high physical water losses resulting from frequent pipeline burst and repairs along water distribution pipelines due to high water pressure heads in aged distribution lines. Pressure management as a way of managing water losses was adopted in this study through reservoir water release level control. The study was carried out at Kimilili water supply scheme managed by Nzoia Water Services Company. The scheme is characterized by aged water distribution network that experiences high pressure variations that are attributed to the steep terrain, aged, inadequate and defective system pressure control devices and increased water demand. This has led to; (i) huge water losses (68% Non-Revenue Water) due to pipeline bursts and leakages, (ii) intermittent water supply in the scheme, (iii) compromised quality of water supplied and, (iv) self-financial unsustainability (82% cost coverage) of the scheme. This study sought to establish the optimal reservoir water release levels for maintaining optimal pipeline network pressures for management of water losses at Kimilili water supply scheme. The study was conducted under guidance of the following specific objectives; To forecast Kimilili water supply scheme water demand up to 2030. To simulate Kimilili water supply distribution network zonal nodal pressures. To establish optimal reservoir water release levels for maintaining minimum allowable zonal nodal pressures. The study targeted all the four categories of all the varying active water consumer connection trends between the years 2008 to 2016. The study also targeted the entire six zonal take off points (nodal) of Kimilili water supply distribution pipeline network. Water demand trend forecast for the years 2017 to 2030 was undertaken using Artificial Neural Network (black box) model while EPANET 2.0 was utilized to hydraulically simulate the nodal point pressures based on the forecasted water demand. The hydraulic simulation incorporated the reservoir at the treatment plant, distribution mains and all the six zonal nodal points where all the six zones connect to the distribution mains. Primary data for the study was collected through field observations using pressure data loggers, a clamp on ultrasonic flow meter and GPS handsets while secondary data was obtained through document review. The study established that; (i) Water demand for Kimilili water supply was increasing with time and the general relationship between time and water demand was defined by a sixth order polynomial function ($y = 9e\text{-}0x^6 - 1e\text{-}05x^5 + 0.0005x^4 - 0.0115x^3 + 0.1178x^2 + 0.1384x + 100.48$). (ii) System water losses decreased with increase in water demand and the general relationship between Kimilili water supply periodic system water demand and system water losses is an exponential function given as ($y = 256394e^{-7.296x}$), while the relationship between periodic system water demand and percentage system water losses is a polynomial function of order two defined by ($y=1.8503x^2 - 21.882x + 88.808$). (iii) Pressure management through optimization of reservoir water release levels for Kimilili Water Supply Scheme may be utilized up to the year 2026, beyond which it might be practically impossible with the current existing infrastructure. The study recommends the water utility to practice both water demand management and system pressure management by utilizing the findings of the study.

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| | |
|---|---|
| AC pipe | – Asbestos Cement pipe |
| ANN | – Artificial Neural Network |
| DDM | – Data Driven Model |
| DMA | – District Metered Area |
| EPS | – Extended Period Simulation |
| FCV | – Flow Control Valve |
| GI | – Galvanized Iron |
| GIS | – Geographical Information System |
| GPS | – Global Positioning System |
| ISO | – International Organization for Standardization. |
| IWA | – International Water Association |
| IWR-MAIN | – Institute for Water Resources Municipal And Industrial Needs |
| LVNWSB | – Lake Victoria North Water Services Board |
| NRW | – Non Revenue Water |
| NSE | – Nash-Sutcliffe Efficiency |
| NZOWASCO | – Nzoia Water Services Company Limited |
| PBV | – Pressure Breaker Valve |
| PRV | – Pressure Regulatory Valve |
| PMAC | – Pressure Monitoring And Control |
| RMSE | – Root Mean Square Error |
| SCADA | – Supervisory Control And Data Acquisition |
| uPVC | – Unplasticized Polyvinyl Chloride |
| UTM | – Universal Transverse Mercator |

WASANIS – Water And Sanitation Information System

WASREB – Water Services Regulatory Board

WSP – Water Service Provider

# CHAPTER ONE: INTRODUCTION

## 1.1 Background of the Study

Potable water supply distribution networks are designed and constructed to convey treated water from the water treatment plant to the end users. The supply distribution system has to meet two primary requirements; First, it needs to deliver adequate amounts of water to meet consumer demand and fire demand requirements. Second, the water system needs to be reliable (availability of required amount of water 24 hours a day in 365 days a year). At the consumer point it is expected that clean water in the right quantity and at the right pressure will be available by just turning on the tap. Performance of water distribution system to consumers' satisfaction is the major challenge for water authorities all over the world. In developing countries, urban water supplies are mostly intermittent, typically ranging from 2 hours to 4 hours in a day due to pipeline burst, repair or maintenance (Ingeduld et al., 2006). Due to high water pressure heads in aged distribution lines, high physical water losses are experienced along water distribution pipelines.

According to WASREB (2015), considering the water sector benchmark of non-revenue water (NRW) of 20%, the average non-revenue water level of water utilities in Kenya stands at 42% which translates to a financial loss of Ksh 5.9 billion to the water sector. This not only threatens the financial sustainability of the sector but also wastes funds which could have been used to increase access and improve service delivery. According to Anil (2004), it is necessary to plan and construct suitable water supply schemes including well designed distribution networks in order to ensure supply of quality potable water in sufficient quantities to the various users in the community in accordance with their demand and requirements. Vasan & Simonovic (2010) suggested the simulation of the water distribution network system by modeling, analyzing, and its performance evaluation through scenario investigation of the physical and hydraulic parameters.

Kimilili water supply scheme of Nzoia cluster is reported to record cost coverage of 82% and the highest Non Revenue Water of 68% as compared to the other three service areas managed under Nzoia cluster. The non revenue water levels for Kitale, Bungoma and Webuye service areas of Nzoia Water Services Company cluster are 35%, 37% and 46% respectively (NZOWASCO, 2015). A baseline survey carried out on non revenue water for Kimilili water supply scheme by Tertiary Consulting Engineers in 2012 established that physical losses contributed to 64% of the schemes' non revenue water while commercial loses contributed to 36%. Furthermore pipe bursts and leakages due to high pressures in the aged distribution pipelines contributed to 63% of the physical losses while pipe vandalism and tank overflows contributed to 37% of the physical losses.

The baseline survey further recommended the scheme to pursue reduction of water losses through implementation of the following; (i) Carrying out water use audits on large volume users in order to identify areas in which overall water use efficiency could be improved through alternative technologies or practices. (ii) Devising and facilitating a consumer retrofit program that would help in promoting water conservation to reduce unnecessary water usage. (iii) Carrying out system pressure management to reduce water loses as well as to ensure longevity of the aging infrastructure. (iv) Establishment of a NRW monitoring unit. (v) Motivating meter readers with adequate training. (vi) Mapping and understanding of the distribution network. (vii) Striving for universal connections metering. (viii) Improving accuracy of meters and meter reading through periodic meter calibration. (ix) Standardization of the water meter brands. (x) Detection and removal of illegal connections. (xi) Carrying out zonal metering. (xii) Carrying out district metering, and (xiii) Replacement of aged pipes in service connections. This research therefore adopted recommendation (iii) as one of its key objectives in the study.

## 1.2 Statement of the Problem

Pressures in gravity driven water pipeline distribution systems vary depending on the elevation differences between pipeline sections of concern in respect to the water reservoir feeding them. The sections of the systems located at lower elevations in relation to the water reservoirs experience higher pressures as compared to the sections of the same systems located at higher elevations. The pressure in the system is usually controlled to ensure it is within a given range to avoid either pipe bursts due to high pressures or trickle of water at consumer point and ingress of pollutants due to negative pressure. Pressure in water distribution system is typically maintained either by a pressurized water tank serving an area, by pumping the water up into a water tower and relying on gravity to maintain a constant pressure in the system, by installing break pressure tanks for maintaining constant pressures on gravitational lines or solely by installing pressure reducer valves. Globally acceptable pressures at the consumer point range from 10 metre head of water to 25 metre head of water. The acceptable water losses for developed countries are 10% while for developing countries are 20%, (WASREB, 2015).

Kimilili water supply scheme pipeline distribution system pressures vary significantly with consumer points at lowest elevations recording as high as 92 metre head while consumer points at the highest elevations recording as low as 8 metre head. The large variations in the distribution network pressures are attributed to the steep terrain, aged, inadequate and defective system pressure control devices and increased water demand. This has resulted into huge water losses (68% NRW) due to pronounced pipeline bursts and leakages in low elevation sections of the system as a result of extremely high pressures. The pronounced pipeline bursts and leakages have in turn led to intermittent water supply in the scheme as a result of shutting down the system during pipe burst and leaks repairs or reduced pressures in highly elevated areas. Pipe bursts and leakages due to high pressures in the aged distribution pipelines contributes to 63% of the physical losses while pipe vandalism and tank overflows contributes to 37% of the physical losses.

If the above performance trends for Kimilili water supply scheme continues, it is feared that the scheme's cost coverage will further drop leading to deterioration of infrastructure in place hence poor quality service provision, which will derail achievement of Sustainable Development Goal 6: "To ensure accessibility and sustainable management of water and sanitation for all by 2030". Implementation of intelligent pressure management system is an efficient approach to be adopted for reduction of water losses resulting from pipeline system damage due to high pressures.

This research therefore establishes demand driven optimal reservoir water release levels for maintaining minimum allowable pipeline network zonal nodal point pressures for management of water losses to enhance continuous water supply at Kimilili water supply scheme by both forecasting the scheme's water demand and simulating the pipeline distribution network system.

## 1.3 Objectives of the Study

The main objective of the study was to establish Kimilili water supply scheme reservoir water release levels that will optimize pressure at the water pipeline distribution network zonal nodal points by forecasting the scheme's water demand and simulating the pipeline distribution network system.

The specific objectives of the study were;-

(i) To forecast Kimilili water supply scheme water demand up to 2030.

(ii) To simulate Kimilili water supply distribution network zonal nodal pressures up to 2030.

(iii) To establish optimal reservoir water release levels for maintaining minimum allowable zonal nodal pressures up to 2030.

## 1.4 Significance of the Study.

The steep terrain of Kimilili water supply scheme has contributed to the schemes' high pressure heads being experienced in the pipeline distribution network system leading to high system water losses of 68% resulting from pronounced pipeline bursts and leakages in low elevation sections of the system due to extremely high pressures. The pronounced pipeline bursts and leakages have in turn led to intermittent water supply in the scheme as a result of shutting down the system during pipe burst and leaks repairs or reduced pressures in highly elevated areas. Consequently there is frequent compromising on the quality of water supplied as result of dirty water entering the distribution network during pipe burst and leak repairs. Ultimately the scheme has become financially self-unsustainable (82% cost coverage) as a result of incurring high expenses during potable water production and distribution process (chemicals costs, power cost, pipeline maintenance costs), at the same time a lot of revenue is lost in terms of the huge water losses incurred of which 64% is contributed by physical losses and 36% by commercial losses respectively (NZOWASCO, 2014/2015).

Through development of water demand forecast model and subsequently establishing optimal reservoir water release levels for maintaining minimum zonal nodal pressures, it is hoped that; The pipeline distribution network will be efficiently sustained with minimum water losses being experienced, the results of this study could be adopted for development of pressure management strategy of Kimilili water supply scheme hence reduction of water losses resulting from extremely high pressures and assurance of continuous water supply as a result of adequate system pressures. The findings of this study will be used by other scholars and researchers to carry out more research on water supply demand management, pressure management and non-revenue water management.

## 1.5 Scope and Limitations of the Study

Artificial Neural Network training required considerable large amounts of data and since gathering data on water demand over time varying trends based on population served was not possible, water demand training, validation and testing data sets were obtained from Kimilili water supply scheme billing system for the period Nzoia Water Services Company had been in operation. The study projected water demand requirements for Kimilili water supply scheme for 4 years up to 2020 being the operational design period of the water supply system. Furthermore the study focused on establishing optimal reservoir water release levels for maintaining minimum allowable zonal nodal pressures for management of bursts and leaks (physical water losses) which were directly influenced by pressure variations in the system, thus did not deal with commercial water losses.

# CHAPTER TWO: LITERATURE REVIEW

## 2.1 Introduction

This chapter discusses literature related to management of water losses in water distribution networks, water demand forecasting and hydraulic simulation. Detailed discussion on water losses concept, water balance concept, water demand forecasting and forecasting models like Artificial Neural Networks has been carried out. The chapter further discusses on simulation models under which hydraulic simulation is greatly explained. The theory of EPANET network analysis algorithm is explained including the relationship between flow and pressure at EPANET emitter node. The chapter finally analyses literature on various studies that have been conducted on use of ANN for forecasting and EPANET for hydraulic modeling and the research gaps.

## 2.2 The Water Losses Concept

Juan (2008), defines water losses as the difference between water produced and the amount of water sold to all customers. It is represented as;

$$Percentage\ water\ loss = \left(\frac{Water\ Produced\ m^3 - Water\ Sold\ m^3}{Water\ Produced\ m^3}\right) x100 \ldots\ldots \text{ Equation (2.1)}$$

There are two main components of water losses, technical and commercial. The first of them emanates from physical failures on the distribution system (pipe leaks and bursts). On the other hand, there is a commercial component that is partly linked to lack of measuring (faulty meters that inaccurately register consumption) and unauthorized consumption of water, (it is water used but not paid for). Unauthorized consumption of water is associated with illegal connections established by users stealing water or taking it without any legal means to measure it or simply by shifting connections in order to lower consumption measurement.

WASREB (2015), defines water loss as the difference between amount of water produced for distribution and the amount of water billed to consumers. Water loss constitutes of real losses (physical) through leaks, apparent losses (commercial) through illegal connections, water theft, metering inaccuracies and unbilled authorized consumption. Water loss is typically measured as the volume of water "lost" as a share of net water produced. However, it is sometimes also expressed as the volume of water "lost" per km of water distribution network per day or volume of water "lost" per connection per day.

### 2.3 The Water Balance Concept

Farley et al. (2008), emphasizes on development of an understanding of the 'big picture' of the water system, which involves establishing a water balance as being the first step in reduction of water losses process. In the United States, water balance is also called 'water audit'. This process helps in understanding the magnitude, sources, and cost of water loss (quantity of water being lost). The International Water Association (IWA) has developed a standard international water balance structure, a concept that has been adopted by national water associations in many countries across the world. Table 2.1 shows the standard international water balance structure developed by IWA.

Lambert & Thornton (2011), international report on ‘Water Losses Management and Techniques’, prioritizes the IWA standard water balance and definitions as the basic and essential first step in management of water losses, then followed by the assessment and management of unbilled authorized consumption which is part of non revenue water, but not part of water losses in the IWA definitions.

**Table 2. 1: Standard IWA water balance structure showing Water Loss components.**

<table>
<tr><td rowspan="9">System Input Volume</td><td rowspan="4">Authorized Consumption</td><td rowspan="2">Billed Authorized Consumption</td><td>Billed metered consumption</td><td rowspan="2">Revenue Water</td></tr>
<tr><td>Billed unmetered consumption</td></tr>
<tr><td rowspan="2">Unbilled Authorized Consumption</td><td>Unbilled metered consumption</td><td rowspan="7">Non-Revenue Water</td></tr>
<tr><td>Unbilled unmetered consumption</td></tr>
<tr><td rowspan="5">Water Losses</td><td rowspan="2">Apparent Losses (Commercial Losses)</td><td>Unauthorized consumption</td></tr>
<tr><td>Metering inaccuracies and data handling errors</td></tr>
<tr><td rowspan="3">Real Losses (Physical Losses)</td><td>Leakage and bursts on transmission and/or distribution mains</td></tr>
<tr><td>Leakage and overflows at utility's storage tanks</td></tr>
<tr><td>Leakage and bursts on service connections up to point of customer metering</td></tr>
</table>

Source: Farley et al. (2008)

Water loss is equal to the total amount of water flowing into the water supply network from a water treatment plant (the ‘system input volume’) minus the total amount of water that consumers are authorized to use (the ‘authorized consumption’). This is demonstrated by equation 2.2.

Water loss = system input volume - billed authorized consumption…….. Equation (2.2)

This equation works on the assumptions that; (i) Any known errors for system input volume are corrected and (ii) Customer billing records for metered consumption period are consistent with the system input volume period.

## 2.4 Water Demand Forecasting

Water demand forecasting is the methodology used to predict future water needs. Gardiner & Herrington (1986), defines forecasting methods as the procedures and conventions used to analyze past water use (explanation) and to apply the resulting knowledge to the future. In other words, forecasting methods translate projected values of one or more of the explanatory variables such as population, income, water price, et cetera into estimates of future water requirements.

Some of the developed forecasting methods are based on an analytical or mathematical approach while others (mainly for short term forecasting) utilize purely heuristic approach (Rahman & Bhagnagar, 1988). Subsequently, some researchers have attempted to integrate both mathematical and heuristic approaches for short term water demand forecasts (Hartley & Powell, 1991).

The success of simulation models results in both an improved understanding of the modeled system and a useful predictive tool (Caswell, 1976 and Rykiel, 1996). The philosophies of Caswell (1976) and Rykiel (1996) suggest that the pursuit of forecasting in research areas such as aquatic sciences improves our understanding of forecast modeling. For example, understanding the variables within forecast models enhances the foundation for simulation science. Rykiel (1996), further argue that the value of simulation and forecast models is determined by the validation process that complements the models.

Successful water demand forecasting depends on many factors including an understanding of the stability of water demand, the availability of essential data, the influences of water demand and how these influences may change in the future. Collecting data and deciding on the format of analysis are critical to the development of a reliable and credible model (Kame'enui, 2003).

Kuczera & Ng. (1993), observed that the inability to predict future sociologic or economic variables that will affect water demand is definitely a limiting factor in simulating future water demands). The forecasts created by agencies on future social and economic conditions contain variables with significant uncertainty, adding error to the water demand forecast analysis, which is also true for climate forecasts. The use of forecasted climate variables in water demand models helps to model water demand under a variety of climate change scenarios with regard to such models. There is a considerable degree of uncertainty associated with climate forecasts. Uncertainties in climate, social, and economic variables are often the result of an inaccurate understanding or downscale of a climate model, as well as unexpected changes in the social and economic structure as a result of cultural trends. These uncertainties are often unpreventable and when combined may result in a model limited by both known and unknown errors and assumptions. Measuring model skill and error may help quantify this uncertainty, however, these measures will not necessarily identify the specific causes of or solutions to model inaccuracy (Boland, 1997).

## 2.5 Water Demand Forecasting Models

Water demand forecasting can be conducted for varying horizons;

a. Short term forecasting aims at anticipating water demand over the coming hours, days or weeks so as to optimize the operation of water systems like reservoirs and potable water treatment plants while factoring in changes in weather and consumer behaviors. Short term demand forecasting can help estimate revenues from water sales and plan short term expenditures.

b. Intermediate term forecasting which ranges between 1 to10 years focuses on the variability of water consumption by a fixed or slowly increasing customer base. It considers changes in the composition or characteristics of the customer base, or economic cycles.

c. Long term forecasting considers horizons of 10 to 30 years. This is the timeframe taken into account when building long lifespan water supply infrastructures such as storages, distribution lines, desalination plants or large capacity inter basin transfers. In long term planning, many factors of change are liable to modify both the customer base and per unit water consumption. Uncertainty is a key issue in long term water demand forecasting (Rinaudo, 2015).

Short and intermediate term operations widely utilize trend based regression time series analysis and artificial neural network techniques while long term demand forecasts are typically derived from an understanding of the requirements of individual end users

(Mazer, 2007). Bauman et al. (1998), classifies water demand forecast models being implemented by water utilities into five main types.

### 2.5.1 Unit Water Demand Based Models

This method is also known as per capita water demand based model, involves determining future water needs as per the number of users, it relies on the use of 'unit water demand' coefficients determined per capita or per unit of industrial output, thus also known as per capita approach / model. Demand is estimated by multiplying these coefficients by the number of users the water utility is liable to serve in the future. The coefficients can be differentiated according to the level of customer disaggregation. The first level of disaggregation generally consists in a breakdown into domestic, commercial, industrial and institutional uses.

Domestic demand may further be decomposed according to housing type, estimating separately multiple dwellings and single family homes and houses with or without meters. Likewise, the demand of industrial and commercial users may be broken down according to activity sector. One can consider the consumption coefficients as variable with time, extrapolating their future direction from past tendencies. This approach is useful where little or no data are available. It may also suffice when a rough estimate is required for preliminary planning purposes. One of its advantages is transparency and so it is easily understood by stakeholders, thus being the most probably used method. Water demand for this model follows the general form:

$q_{c,m,l} = (Q_{c,m}/N_c n_c)$ ………………………………………………………… Equation (2.3)

Where:

$q$ = *average use per capita (litres per capita per day)*

$c$ = *customer class*

$m$ = *month*

$l$ = *location (i.e., county)*

$Q$ = *water consumption (litres)*

$N$ = *number of accounts*

$n$ = *average number of people served per account.*

### 2.5.2 Multivariate Statistical Models

This model recognizes that change in demand arises from varying factors, including water tariffs, household size, household income, economic activity, climate, water policies among others. The method characterized by estimating the statistical relationship between per capita consumption which is dictated by a set of explanatory variables (varying factors). The model is generally built using panel data that is water utilities for which data of 5–10 year interval is available. The model can be used for prediction purposes to calculate the demand that would be obtained under a hypothetical evolution of the explanatory variables, supposing that the model coefficients hold true over the future time window considered. (Bauman et al., 1998; Dalhuisen et al*.,* 2003 and Rahman & Bhagnagar, 1988), supports the development of this model type. Fullerton & Molina (2010), observed that the main weakness of statistical models for long range forecasting is their out of sample predictive capacity. The water demand follows the general form of Equation 2.4.

$q_{u,c,m,y} = \alpha\ (x1^{\beta 1} x2^{\beta 2} \ldots xn^{\beta n})_{u,c,m,y}$ …………………………...……...……. Equation (2.4)

Where

*q = per unit use or house hold use (litres per household)*

*u = utility (category of water service provider – small, large, very large)*

*c = customer class (category of the customer – domestic, industrial, commercial)*

*m = month*

*y = year*

α = *intercept*

*X = explanatory variable*

*β = elasticity*

### 2.5.3 Data Driven (Black Box) Models

The assumption that the future evolution of demand can be deduced from past tendencies is the basis of this modeling approach. (Butler & Memon 2006; McMahon 1993, Wurbs 1997) concurs that under this approach, several techniques including; Fuzzy Logic, Artificial Neural Networks, Expert Systems, Kalman Filter and other techniques are used to forecast future water use based on recent or historical water consumption. The projection of the tendencies may be applied locally at the scale of a single drinking water utility or of a region or can even be refined by reasoning according to types of consumers. Sophisticated geostatistical methods that simultaneously consider time and space variability have also been used to map future water demand. With extrapolation approach, the only data required are time series of the variable being forecasted. However, its predictive capability is quite limited because it is unable to take into account changes in the socioeconomic context (employment, tariffs, urban patterns and population) and the occurrence of discontinuities like changes in technology (Dalhuisen et al. 2003).

### 2.5.4 Micro Component Models

This approach assesses total consumption by simulating in detail variations in the ways consumers use drinking water. It is also known as 'end use modeling' and is applied majorly for domestic demand forecasting. The approach estimates the amount of water associated with each of the main water use devices: kitchen taps, lavatories, bathtubs, showers, sanitary facilities, household appliances and outdoor devices. Each use is the product of (i) device ownership percentage, (ii) frequency of use and (iii) volume per use. This approach is mainly advantaged in that it enables the long term effect of technological evolution to be simulated (decreased volume of toilet flush, appliance performance). Rinaudo (2015), confirms that micro component models are more prospective thus allows the effects of water conservation policy incentives to be estimated. The method is widely used in United States of America (Levin et al. 2006), United Kingdom (Thames Water, 2010) and South Africa (Jacobs & Haarhoff, 2004) water industries. Micro component models are generally represented by Equation 2.5.

$q_{c,m,y} = (Qb/Nb)c,m\ (X1f/X1b)^{\beta 1}c,m\ (X2f/X2b)^{\beta 2}c,m \ldots (Xnf/Xnb)^{\beta n}c,m$ . Equation (2.5)

Where

$q$ = *adjusted per unit use*

$c$ = *customer class*

$m$ = *month*

$y$ = *year (b = base period; f = future year)*

$Q_b$= *base year per unit use*

$N_b$= *counting unit (e.g., account, housing unit, population, etc.)*

$X_b$ = *base year factor variable*

$X_f$ = *projected factor variable*

$\beta$ = *elasticity*

### 2.5.5 Composite Models

These are hybrid models combining two or more of the four methods described and many of them are applied by water utilities. This is also the case for water demand forecasting software packages such as Institute for Water Resources Municipal And Industrial Needs (IWR-MAIN), which has been intensively used in the USA (Bauman et al., 1998 ) includes a variety of forecasting models including; extrapolation models, statistical models, unit water demand models, and end use models. (Mohamed and Al-Mualla, 2010), observes that this software has been used by more than 40 large American cities and state organisations and elsewhere around the world. A number of other hybrid tools have been developed and tested as part of research projects such as the demand forecasting and management system (Froukh, 2001).

### 2.5.6 Analysis of Water Demand Forecasting Models

Unit water demand forecasting models are mostly applied in development of sectoral water demand forecast accounting for expected future population growth, change in economic activity per branch. The demand can easily be represented spatially and can be linked to GIS data. The model utilizes data of both unit water consumption coefficients and estimated future number of water users per consumption category. The major shortcoming of unit water demand forecast model is that it does not account for possible future changes in unit water consumption due to evolving water tariffs and household income. The validity and reliability of the forecast data is low as varying factors are not taken in account by the model (Rinaudo, 2015).

Multivariate statistical water demand forecast models are applied in forecasting future water demand by considering changes in population and economic activity and changes in socioeconomic variables like water rates, households' characteristics and income. The model utilizes data of time series for water consumption and all explanatory variables and the estimated future number of users per water consumption category. The model does not account for changes in plumbing code or campaigns to promote water conservation (Ji et al, 2012). Development of the model requires considerably more time to do the surveys and the forecasts.

Micro-component water demand forecast models are used in forecasting of water demand considering future changes in household appliances and indoor or outdoor water use practices. They are also applied in ex-ante evaluation of the efficiency of water conservation policies. For this model, data is required on wide scale households' survey to assess customer appliance ownership, frequency of use, and volumes of water used. The model is mainly adapted to residential water demand and is often used in combination with a multivariate statistical model. The model requires considerable more time to develop and to do the forecasts (Rinaudo, 2015).

Comparing the water demand forecasting models, the data driven models (DDMs) are relatively easy to construct and produces excellent estimates, although they have the disadvantage of lacking mathematical and physical logic and they are unable to take into account changes in the socioeconomic context. DDMs are relatively quicker to develop and easier to use when compared to the process-based models, DDMs have the ability to directly define input output mapping functions relationships, thus the large computational and data requirements often associated with process-based models are to some extent reduced in DDMs.

The use of DDMs has also been seen as a promising technique to solving the sensitivity and uncertainty challenges inherent in the use of process based models. Artificial Neural Networks (ANN) is a data driven forecasting model technique that has the ability to discover input-output mapping functions, thus it has extrapolation ability when presented with unfamiliar input vectors, meaning, it has the ability to predict values higher than those in the range of the historic observations. This serves as a major advantage to the use of ANN, as it is credible when used for real forecasting (Oluwaseun et al, 2014).

The ANN model makes rules with training data, constructs a model, and provides forecasts based on the observed water demand data. Overall, the ANN model provides more accurate estimates. It can be considered that the ANN model as a supplementary tool to the physical model could help to reduce uncertainties and address problems of the current water demand forecast and warning system, as well as improving the accuracy of demand forecasts. Off-shelf ANN software's are also available for tailor making, this makes ANN the most appropriate water demand forecast model technique to adopt for medium term water demand forecast (Santos and Augusto, 2014).

## 2.6 Artificial Neural Networks

Smith, (1993), defines Artificial Neural Networks (ANN) as a data driven (black box) forecasting model technique. ANNs are massively parallel distributed, adaptive, generally nonlinear networks built from many different processing elements (nodes) that processes information by simulating the working of the neuron network in human brain. The neurons are responsible for the human learning capacity and this significant property is used in machine learning in artificial neural networks. Perea et al. (2015), defines a neural network as a system that allows for linear or nonlinear relationship between outputs and inputs. Its main features are inspired in the nervous system which gives them several advantages such as to have adaptive learning ability, to be self-organizing, to be able to operate in parallel in real time and to provide fault tolerance by redundant information coding.

The basic structure of ANNs consists of an input layer of nodes that receive external inputs, hidden layers and an output layer. The nodes are generally arranged in layers which provide an information flux from input layer to output layer. The input layer consists of nodes that receive an input from the external environment. These nodes do not perform any transformations upon the inputs but just send their weighted values to the nodes in the immediately adjacent, usually 'hidden,' layer. The hidden layer(s) consists of nodes that typically receive the transferred weighted inputs from the input layer or previous hidden layer, perform their transformations on it, and pass the output to the next adjacent layer, which can be another hidden layer or the output layer. The output layer consists of nodes that receive the hidden layer output and send it to the user.

The number of input nodes in an input layer corresponds to the number of input variables while the number of nodes in an output layer corresponds to the number of outputs (Loucks & Beek, 2005). There can be several hidden layers between input and output layers, the hidden layers increase the network's ability to model more complex events. There are two major connection topologies that define how data flows between the input, hidden and output nodes; *Feed forward networks* in which the data flow through the network in one direction from the input layer to the output layer through the hidden layer(s). *Recurrent or feedback networks* in which the data flow not only in one direction but in the opposite direction as well for either a limited or a complete part of the network. Figure 2.1 shows a typical multi-layer Artificial Neural Network with four input layers, one hidden layer and one output layer.

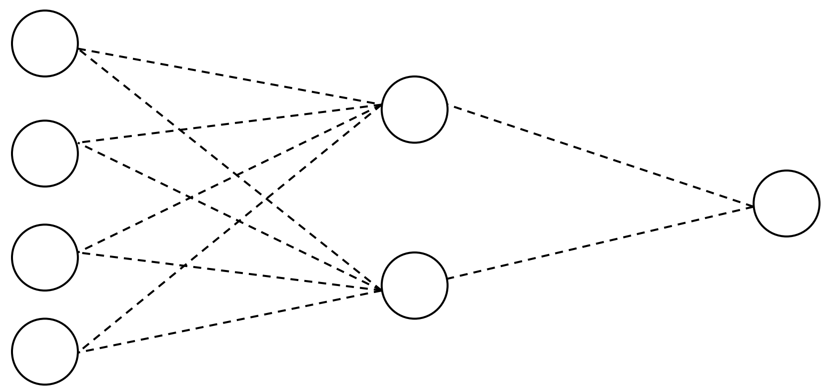



**Figure 2. 1: A typical multi-layer artificial neural network showing the input layer for four different inputs, the hidden layer and the output layer having one output.**

An ANN does not have any knowledge at the beginning, learning process starts on entering data into the input layer of the network. The error back propagation (BP) algorithm is used to adjust the interconnection weights during training. Typically, ANNs preparation and training requires three data sets; training data set for training the network, validation data set for network error validation and testing data set for testing the trained network model (Loucks & Beek, 2005).

Since the introduction of the artificial neurons concept by McCulloch & Pitts (1943), major applications of ANNs have arisen after the development of the error back propagation method of training. An application of ANN models in research has resulted in successful solution of some complicated problems not easily solvable by traditional methods (Altunkaynak, 2006). It is demonstrated that ANNs are robust tools for modeling many nonlinear hydrologic processes such as rainfall runoff, stream flow, ground water management, water quality simulation and precipitation. Stochastic models of water demand are based on coefficients of water demand time series and meaningful variables, alternatively, artificial neural network models can handle nonlinear processes and can be adjusted (Santos & Augusto, 2014). Simple models cannot handle adduction nonlinearities and the complexity of the water supply systems, so an ANN is a better model for decision making since it takes into account nonlinear processes related to inputs and outputs of a system.

Altunkaynak, (2006), study on forecasting of water level fluctuations in Lake Van in eastern Turkey for five years test period prediction concluded that Artificial Neural Networks models outperformed Autoregressive Moving Average (ARMAX) models. Perea et al., (2015), developed a combined ANNs and genetic algorithm featured forecasting model of the daily water demand for irrigation which was applied to predict water demand one day ahead in the Bembézar M.D. Irrigation District (BMD) in Southern Spain. The model predicted 93 % of the variability of the observed water demand with a standard error of 12.63 %, which established that the methodology improved the accuracy of the predictions of previous models with smaller errors. Further studies carried out by Santos & Augusto (2014), on water demand forecasting for the Metropolitan Area of São Paulo in Brazil by means of ANN model with the input of weather information with hourly time resolution established that ANN model yielded better training and prediction performance than the multiple linear programming (MLR) model.

The suitability of ANNs applications in water demand forecasting is also demonstrated by Cubero, (2012), by developing and applying two forward neural networks model (with and without intervention series) for obtaining single step predictions of the daily demand for a large water network for Barcelona in Spain. The resulting error was of the same order as that obtained with the Box-Jenkins method despite the fact that no previous filtering or statistic treatment had been performed on the data used. The study established that the neural networks methodology had low requirements of statistics knowledge and data pre-processing and through on line learning algorithms, it could work on line integrated into centralized remote control systems like EPANET.

Heller & Wang, (1996), study for forecasting seven year long time series of water demand for the municipality of Syracuse in Italy, a Hybrid Neural Networks model was used to forecast water demands to improve the accuracy of potable water demand forecasts. The hybrid method of forecasting proved to be superior to conventional linear forecasting tools and to pure neural networks. The hybrid neural network showed marked improvement in interpreting and training complex data sets because it was able to identify seasonal lags. The time required to train and test the neural network using the hybrid method was reduced significantly because it allowed the use of smaller and more appropriate network input structures. The neural network forecasting can operate with Supervisory Control And Data Acquisition (SCADA) systems, permitting water utilities to have access to real time integrated water resources management information.

### 2.7 Model Calibration and Validation

In hydraulic modeling, there are two main exercises that must be successively achieved before using a model. These are calibration and validation of the model. Calibration is an iterative exercise used to obtain the most suitable parameter in modeling studies. The exercise is very vital as reliable values for some parameters can only be found by calibration.

The model parameters to be changed during calibration are majorly classified into physical and process parameters. Physical parameters represent measurable properties of the water supply scheme such us pipeline network length and elevations of the pipeline points while process parameters represent water supply scheme characteristics that are not directly measurable such as water losses and cost coverage. These require prior knowledge of the water supply scheme properties and behavior to be able to specify the initial parameters of the model. There are three calibration methods that can be applied in modeling; manual, automatic and a combination of the two (Tabesh et al, 2011). Manual calibration involves parameter assessment through a number of simulation runs. A good graphical representation of the simulation results is a prerequisite for this method. It is subjective to the modelers' assessment and can be time consuming.

Automatic calibration involves use of system inbuilt numerical algorithm which find extreme of a given numerical objective function. Automatic calibration searches through as many combinations of experimental parameter levels as possible. The method is fast and less subjective. A combination of the two methods involves initial adjustment of parameter values by trial and error to delineate rough orders of magnitude of the parameters followed by a fine adjustment using automatic optimization within delineated range of physical realistic values.

Model validation is the process of demonstrating that a given site specific model is capable of making sufficiently accurate simulations. This involves application of the model without changing the parameter values that were set during calibration. There are four hierarchical schemes for systematic validation of hydraulic models; split sample test, differential split sample test, proxy basin sample test and proxy basin differential split sample test (Refsgaard, 1996). The model is said to be validated if its accuracy and predictive capability in the validation period have been proven to lie within acceptable limits (Tabesh et al, 2011).

### 2.7.1 MATLAB

MATLAB (matrix laboratory) is a software that has an engine tool box for running neural network simulation, is a multi-paradigm numerical computing environment and proprietary programming language developed by MathWorks. MATLAB performs simulation through matrix manipulations, plotting of functions and data, implementation of algorithms, creation of user interfaces, and interfacing with programs written in other languages, including C, C++, Java, Fortran and Python. The MuPAD symbolic engine tool box allows access to symbolic computing abilities, whereas Simulink toolbox adds graphical multi-domain simulation and model-based design for dynamic and embedded systems. The MATLAB application is built around the MATLAB scripting language. Common usage of the MATLAB application involves using the Command Window as an interactive mathematical shell or executing text files containing MATLAB code. Data is uploaded into MATLAB in form of variables referred to as structure array

MATLAB supports developing applications with graphical user interface (GUI) features, it also has tightly integrated graph-plotting features. Neural network functions are executed though the neural network toolbox (nntool) of MATLAB.

### 2.8 Model Performance Assessment

During calibration and validation of the water forecasting and hydraulic simulation models it is necessary to assess the performance of the models. This is achieved by statistically comparing the model (predicted) values with the observed values using various statistical measures which include; the Coefficient of Determination ($R^2$), Root Mean Square Error (RMSE) and Nash-Sutcliffe Efficiency (NSE). The measures are statistically expressed as shown in Equations 2.6 to 2.8.

Coefficient of Determination ($R^2$);

$$CD = \frac{\sum_{i=1}^{n}(O_i - \bar{O})^2}{\sum_{i=1}^{n}(P_i - \bar{O})^2} \quad \text{.......... Equation (2.6)}$$

Root Mean Square Error (RMSE);

$$RMSE = \left[\frac{\sum_{i=1}^{n}(P_i - \bar{O})^2}{n}\right]^{0.5} \cdot \frac{100}{\bar{O}} \quad \text{.......... Equation (2.7)}$$

Nash-Sutcliffe Efficiency (NSE):

$$NSE = \frac{\left(\sum_{i=1}^{n}(O_i - \bar{O})^2 - \sum_{i=1}^{n}(P_i - O_i)^2\right)^2}{\sum_{i=1}^{n}(O_i - \bar{O})^2} \quad \text{.......... Equation (2.8)}$$

Where;

$P_i$ = predicted value.

$O_i$ = observed value.

n = number of samples.

$\bar{O}$ = mean of observed data.

The $R^2$ is a measure of the proportion of the total variance of observed data explained by predicted data, a perfect fit also being one with a lower limit of zero and an upper limit of infinity. It tells us whether the model is over predicting (a value under one) or under predicting (a value over one). RMSE is the root of mean square error expressed as a percentage of the observed mean. That is the average error of predicted results and the mean error is the single greatest error between predicted and observed results. The NSE tells us how well the model is performing in prediction, a value of one indicates a perfect one-to-one relationship and any negative value tells us that the model is worse at predicting observed data than when using the mean of observed values to predict the data.

### 2.9 Simulation Models

Simulation models addressing '*what if*' questions, are used to establish what will likely happen over time and at one or more specific places if a particular design and or operating policy is implemented.

#### 2.9.1 Types of simulation models

Simulation models can be categorized into; statistically oriented, process oriented or hybrid models. Pure statistical models are based solely on data (field measurements) while pure process oriented models are based on knowledge of the fundamental processes that are taking place, they incorporate and simulate the physical processes taking place in a system. Static model tries to determine the values of the variables of the system for a given situation, without taking into account variation with time of the parameters of the system. Hybrid models are based on both field measurement data and physical processes taking place in a system. The values of parameters in process oriented and hybrid models are estimated through model calibration, which is usually done using measured field data.

The simulation models in which the external environment of the system being simulated does not change are called static or stationery models, they simulate some particular time in the future where future conditions such as demands and infrastructure design and operation are fixed. The models in which the external environment of the system being simulated changes over time are referred to as dynamic, they simulate developments over time example being decreasing reservoir storage capacities over time due to sediment load deposition or increasing costs over time due to inflation.

Simulation models can also be categorized as either deterministic or stochastic. A simulation model is referred to be deterministic when a given input always produces the same output. The variable of the model take deterministic values hence the insignificant input elements (noise elements) are neglected from the model as they have little or no influence to the output. The case where a given input to a simulation model produces varying outputs is referred to as stochastic simulation model, the model is also significantly influenced by some internal variables.

### 2.9.2 Hydraulic Simulation

Numerous computer models (computer software) have been developed to solve the network hydraulic simulation equations. One of the more widely used models is EPANET software developed by the USA Environmental Protection Agency, EPANET was adopted in this paper because it has many advantages that make it useful for hydraulic analysis of water transmission and distribution networks compared to the other alternative tools available on market; It is for general public and educational use and it is freely accessed via internet. It has an integrated environment for editing network input data and viewing the results in a variety of ways such as graphical format. The nodal "emitter" function of EPANET can be utilized to carry out leakage modeling. The software model can simulate steady state conditions, extended period simulations of hydraulic and water quality behavior within pressurized pipe networks.

EPANET requires relatively small computer space to operate and has unlimited number of pipes that can be analyzed. The user's manual to guide the users in understanding the software is downloaded free. These are obvious advantages for students, researchers and professionals of the developing economies who may not have the financial means to acquire other sophisticated tools (Rossman, 2000). EPANET has become a popular tool in analyzing complex and simple water distribution networks in both the developed and developing countries of the world.

The hydraulic modeling capabilities of EPANET are;

i. Utilizes the Hazen-William, Darcy-Weisbach and Chezy-Manning formula in computing friction head loss.
ii. Has no limitation on the size of the network to be analyzed
iii. Models constant or variable speed pumps
iv. Includes minor head losses for bends, fittings, etc.
v. Allows storage tanks to have any shape and size.
vi. Computes pumping energy and cost
vii. Models various types of valves including pressure regulating, shutoff, flow control and check valves.

viii. Considers multiple demand categories at nodes, each with its own pattern of time variation.
ix. Can perform system operation on both simple tank level and timer controls and on complex rule based controls.
x. Models pressure-dependent flow issuing from emitters (sprinkler heads).

The simulation capabilities of EPANET have been utilized by both professionals and researchers in the design, operations and improvement to various water network distribution systems. Based on the above advantages, Artificial Neural Networks forecasting model and EPANET simulation model were adopted for this research.

### 2.9.3 Theory of EPANET Network Analysis Algorithm

The purpose of a water distribution pipe network system is to supply water at adequate pressure and flow. Pressure is lost by the action of friction at the pipe wall and static head, the pressure loss is also dependent on the water demand, pipe length and diameter. Several established empirical equations describe the pressure-flow relationship (Rossman, 2000). These equations have been incorporated into EPANET network modeling software and the algorithm is briefly described here.

The main principle of EPANET network analysis is based on the continuity equation and conservation of energy theory. The continuity equation implies that the algebraic sum of the flow rates in the pipes meeting at a node together with any external flows is zero. This is illustrated in Figure 2.2 and Equations 2.9 and 2.10.

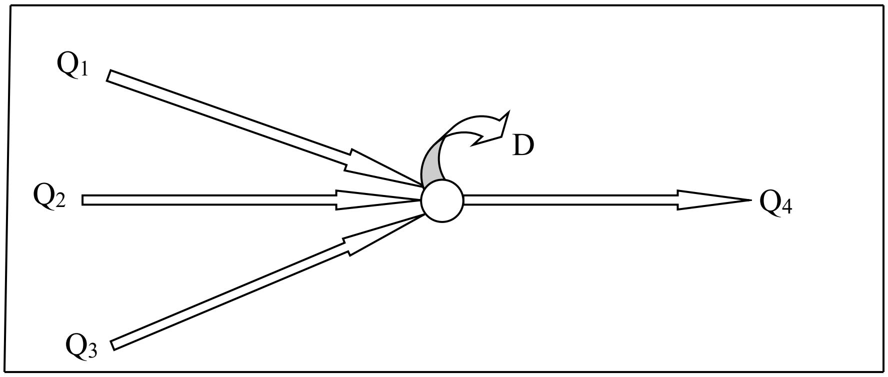



**Figure 2. 2: Continuity Equation Diagram.**

Source: Oyelowo & Adeniran, (2013).

$Q_1 + Q_2 + Q_3 = Q_4 + D$ ………………..……………………….……… Equation (2.9)

$D = Q_1 + Q_2 + Q_3 - Q_4$ ……………………………………………….... Equation (2.10)

Where;

Q = Flow in or out of the node ($m^3/s$)

D = Demand at the node or nodal demand ($m^3/s$).

The conservation of energy condition implies that, for all paths around closed loops and between fixed grade nodes, the accumulated energy loss including minor losses minus any energy gain or heads generated must be zero. This is illustrated by Figure 2.3 and further supported by Equation 2.11.

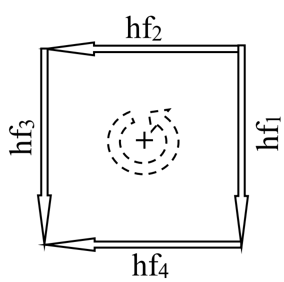



**Figure 2. 3: Part of a Network to illustrate Conservation of Energy.**

Source: Oyelowo & Adeniran, (2013).

Given total head loss for each link (pipe) as $h_f$ and assuming counter clock wise flow direction to be positive, then:

$- h_{f1} - h_{f4} + h_{f3} + h_{f2} = 0$ ……………………………………………............ Equation (2.11)

The hydraulic head lost by water flowing in a pipe due to friction with the pipe walls can be computed using one of three different formulas:

• Hazen-Williams formula

• Darcy-Weisbach formula

• Chezy-Manning formula

The Hazen-Williams formula is the most commonly used head loss formula in the world. It cannot be used for liquids other than water and was originally developed for turbulent flow only. The Darcy-Weisbach formula is the most theoretically correct. It applies over all flow regimes and to all liquids. The Chezy-Manning formula is more commonly used for open channel flow.

Hazen-Williams head loss equation is given by;

$h_f = 10.69[q/C_{HW}]^{1.852}d^{-1.487}L$ …………………………………………... Equation (2.12)

The Darcy-Weisbach head loss equation is given by;

$h_f = 0.0252f(\varepsilon,d,q)d^{-5}L$ ………………………………………........…… Equation (2.13)

Chezy-Manning head loss equation is given by;

$h_f = 4.66n^2d^{-5.33}L$ ……………………………………………………...….… Equation (2.14)

Where;

$h_f$ = head loss (m),

L = pipe length (m),

d = pipe diameter (m),

q = flow rate in the pipe ($m^3/s$),

$C_{HW}$ = Hazen-William Coefficient.

ε = Darcy-Weisbach roughness coefficient.

f = friction factor (dependent on e, d, and q).

n = Manning roughness coefficient.

Each formula uses Equation 2.15 to compute head loss between the start and end node of the pipe:

$h_L = Aq^B$ ………………………………………………………………… Equation (2.15)

Where;

$h_L$ = head loss (m),

A = resistance coefficient,

q = flow rate (Volume/Time),

B = flow exponent.

**2.9.4 Relationship between flow and pressure at an emitter node.**

Emitters are devices associated with junctions that model the flow through a nozzle or orifice that discharges to the atmosphere, they are used to model flow through sprinkler heads or pipe leaks. The flow rate through the emitter varies as a function of the pressure available at the node as provided in Equation 2.16.

$P = Cp^{\gamma}$ ……………………………………...………………..…… Equation (2.16)

Where;

$q$ = flow rate.

$p$ = pressure.

$C$ = discharge coefficient,

$\gamma$ = pressure exponent.

Emitters at junctions are modeled as a fictitious pipe between the junction and a fictitious reservoir. The pipe's head loss parameters are; $n = (1/\gamma)$, $r = (1/C)^n$, and $m = 0$ where C is the emitter's discharge coefficient and γ is its pressure exponent. The head at the fictitious reservoir is the elevation of the junction. The computed flow through the fictitious pipe becomes the flow associated with the emitter (Rossman, 2000). Rossman (2007), suggested that given the relationship between the flow and pressure at an emitter node, pressure dependent analysis of water distribution systems could be accomplished using emitters. The author further provided equation 2.17 to be the general equation for the flow at an emitter.

$q_j^{avl} = C_d\left(H_j^{avl} - H_j^{\min}\right)^{\gamma} : H_j^{avl} \geq H_j^{min}$ ………………………… Equation (2.17)

Where;

$q_j^{avl}$ = flow at demand node $j$.

$Cd$ = Discharge Coefficient

$H_j^{avl}$ = Head at demand node $j$.

$H_j^{min}$ = Minimum head required to cause flow at demand node $j$.

ɣ = Empirical exponent

EPANET application in water transmission and distribution networks hydraulic simulation analysis problems have been reported by many researchers and scholars; Dave et al. (2015), carried out a study on continuous water distribution network analysis using Geo-informatics technology and EPANET in Gandhinagar City, Gujarat state, India. The study indicated that the outcome result from EPANET 2.0 software for pressure, head loss and flow rate were in agreement with hydraulic equation calculations, thus could be used for modeling the water distribution system in Gandhinagar city. EPANET software was successfully applied by (Mohapatra et al., 2014) to simulate intermittent supply system using artificial reservoir approach pilot study area in Nagpur city, India. Mohamed and Abozeid (2011), study on **'**Dynamic Simulation of Pressure Head and Chlorine Concentration in the City of Asyut Water Supply Network in Abnormal Operating Conditions' using EPANET software established that Leakage in the networks not only increases the consumed discharge and decreases the pressure head through the network but also changes the flow directions in some pipes.

Nazif et al. (2009) utilized EPANET 2.0 to simulate hydraulic characteristics of Tehran water distribution network. The results show the importance of the integration approach of EPANET 2.0 software and ANN technique as a simulation tool for optimum operation of Tehran water distribution network. In a study carried out by (Mohamad et al. 2013) to determine the effect of water pressure to water loss in water distribution network of Universiti Teknologi MARA, Shah Alam, Selangor in Malaysia, the results showed that there was slightly small difference between actual and simulated values. Thus EPANET 2.0 is a reliable software model for carrying out hydraulic simulations in a water distribution network. EPANET software model is widely applied in planning of water transmission and distribution infrastructural network development.

Basing on Beijing reclaimed water utilization planning, the multi-sources reclaimed water network integrated hydraulic model was developed based on EPANET and GIS. The model provided strong support to the reclaimed water network planning and management. The adjusted network which was based on the hydraulic simulation provided the scientific basis for the built Beijing reclaimed water pipe network of which the scheme was adopted by Beijing municipal government in China (Jia et al. 2008). Karadirek et al. (2012) undertook a study on pressure management to reduce water losses in a water distribution network in Antalya, Turkey using minimum night flows (MNF) combined with a hydraulic model in EPANET 2.0 and optimized pressure settings of PRVs. The study concluded that hydraulic modeling is essential for applying appropriate pressure management strategies.

Samir et al. (2017), presented modelling leakage as a function of pressure and pipe length, calibration of leakage coefficient using fixed pressure reducing valves (PRVs) to develop pressure fluctuation and EANET scenarios. It is applied to the District Measured Area (DMA) in Alexandria, Egypt. The application of this produced some results, the leakage through DMA is dropped by 37%. It is concluded that the EPANET hydraulic simulation program is used to run different leakage scenarios and Infrastructure leakage index (ILI) showed its performance in evaluating the network and leakage reduction. Tijjani, (2015), successfully designed a water distribution network for Kano Metropolitan in Nigeria using EPANET and the system was able to produce results for all the required design parameters.

Kumar and Karthik (2014), study for analyzing PSNA College of Engineering and Technology, Dindigul water distribution network and identify deficiencies in its analysis, implementation and its usage using EPANET established that the resulting pressures at all the junctions and the flows with their velocities at all pipes were adequate enough to provide water to the study area. Comparison of the pipeline pressure and head-loss which were calculated both manually and software based showed that the results were nearly equal in both the calculation.

Lungariya et al. (2016), utilized EPANET to develop a water distribution system model for analysis of continuous water distribution in Surat City. The study established that the flow computed using EPANET was nearly equal to the actual flow, the velocity computed using EPANET was nearly equal to the actual velocity and head-loss computed using EPANET was nearly equal to the actual head-loss. Comparison of the results indicated that the simulated model seemed to be reasonably close to actual network.

### 2.9.5 Water Losses and Network Pressure Management

Theoretically, water leakage from the distribution network occurs when the residual resistance of the pipe can no longer bear the impact of water pressure. Therefore, approaches for water leakage control can basically be classified into two categories; improving pipe resistance and reducing water pressure. The first category focuses on pipes conditions, breaks are detected and repaired, and deteriorated pipes are repaired or replaced. Thus, the condition of the distribution network can be improved and water leakage can be reduced (Nazif et al. 2009). However, there are many leaks that cannot be detected with available technologies. Pipe replacement can further reduce water leakage, but replacement cannot be implemented on a massive scale due to high costs and long implementation times (Xu et al. 2014).

The second category focuses on water pressure management. Water leakage is positively related to water pressure, and reduction in water pressure can be translated into reduction in water leakage. The total leakage in a pipe distribution network is often estimated according to the pressure leakage relationship in the following form (Lambert 2000; Lambert & Thornton 2011);

$L= K\acute{P}^{n}$ ………………………………………………………………… Equation (2.18)

Where;

L = leakage.

$\acute{P}$ = average pressure of the network.

k and n = calibration parameters.

The exponent n ranges from 0.5 to 2.5 or even higher depending on the type of leakage and pipe material (Farley et al, 2008; Lambert & Thornton 2011). For leaks from joints of fittings, bigger values of n (>1) are usually obtained. For leaks from holes in pipe, n usually has smaller values. Regarding to the pipe material, plastic pipes have bigger n values than metal pipes (Lambert, 2000).

### 2.9.6 Pressure Management

Pressure management is an effective strategy to reduce water leakage in distribution networks. Furthermore, it is the only strategy that allows for reductions in residual water leakage due to undetectable pipe damage. In addition to reducing water leakage from existing pipe breaks, pressure management also reduces the risk of new breaks and extends pipe lifetime (Farley et al, 2008). Pressure management is achieved either through installation of mechanical control devices in the network or control of network inlet pressure. The mechanical control devices which include Pressure Regulatory Valves ( PRVs) and Actuators are mainly installed in networks to reduce too high incoming pressure from water mains to lower (preset values) more functional water pressure by releasing excess energy bound water from the network. Surge tanks are also mechanical devices installed at the downstream ends of feeders on pumping networks to absorb sudden rises of pressure, as well as to quickly provide extra water during a brief drop in pressure.

Control of network inlet pressure is mainly achieved through pump regulation on pumping systems or through reservoir (feed tank) water level regulation in gravity fed systems. Break Pressure Tanks are used to control network inlet pressure in gravity driven distribution networks by reducing the static pressure in the pipe flow to atmospheric pressure, are usually installed for every 100m of elevation change in the network or for tap stands with greater than 20-30m of water head (Jones, 2011).

Apart from network inlet pressure control through reservoir (feed tank) water level regulation technique, all the other mentioned pressure management techniques requires additional investment costs as they usually require to be incorporated in the initial system design. Subsequently the system needs to be redesigned / modified for their inclusion. Thus they are expensive and require expertise. Network inlet pressure control through reservoir (feed tank) water level regulation is an operational issue and neither does it require system modification nor expertise, implying it is less costly to implement thus the basis of this study adopting it.

Studies have shown that pressure management is an important measure for the reduction of real losses which happen through pipe burst due to high pressures (Lalonde et al. 2011). A technique for leakage reduction is pressure management, which considers the direct relationship between leakage and pressure. To control the hydraulic pressure in a water distribution system, water levels in the storage tanks should be maintained as much as the variations in the water demand allows. The problem is bounded by minimum and maximum allowable pressure at the demand nodes.

Nazif et al. (2009) developed and simulated a pressure management model for Tehran water distribution network with an integrated storage facility in the northwest part of

Tehran Metropolitan area using EPANET 2.0 software and ANN technique. The results show that pressure management can be achieved through tank water level regulation, also network leakages due to high pressures can be reduced more than 30% during a year when tank water level is optimized by the proposed model.

Tabesh et al. (2009) utilized EPANET 2.0 to simulate network leakage values and considered the effects of leakage on the pressure of the demand nodes in a water distribution system. The results show that this method can be effectively used for modelling nodal leakage in water distribution networks.

An empirical analysis of the University of Lagos water distribution network in Nigeria was carried out using EPANET software by (Oyelowo & Adeniran, 2013) to establish the optimal tank operational elevation levels for maintaining sufficient pressure heads at the nodal points. The results indicated that the nodal point's sufficient pressure heads and adequate velocities in the network pipeline would be attained by operating the tank at an elevation of 38.549m instead of 8.549m that it was being operated at. Araujo et al. (2006) utilized a hydraulic simulation model, EPANET and two other operational models to manage pressure levels in water distribution systems for the purpose of leak reduction.

Xu et al. (2014) conducted a large scale real world experiment to establish the pressure leakage relationship in a district metering area (DMA) of a water distribution network of Beijing, China. From the experiment, it was found that flow was considerably more sensitive to pressure than expected. Average flow decreased from 31.2 l/s to 16.9 l/s (or 46 %) after pressure management.

A study conducted by (Motiee, 2007) to quantify physical water losses in a distribution network for Ghazvin City pilot area utilizing GIS and EPANET models using Germanopoulos method showed that; the total loss of water was 9168m$^3$ in 28 days and as a result approximately 13% of water supplied had been lost because of high pressure in the pipelines. The study further concluded that to reduce the amount of physical water losses, it was necessary to replace old pipes with high quality long life pipes and to use better joints that can withstand periods of high pressure.

Based on all these studies highlighted, it is evident that larger portion of water losses experienced in water distribution networks is a consequent of high pressures in the pipeline networks. The studies also concurs that pressure management is one of the most efficient ways of reducing physical water losses levels in water distribution networks especially inlet pressure control which is an operational measure that does not require further major investments in the existing water distribution system. Furthermore, hydraulic simulation through use of EPANET software is among the most reliable methods of developing both network mechanically controlled device and network inlet pressure management models. All these studies among others which have been documented have been done on short term basis, little is known on simulation of future water distribution network pressure management models based on future water demands. Thus, this study explored future water distribution network nodal points inlet pressure management through EPANET simulation based on middle term water demand forecasting employing artificial neural networks approach.

### 2.10 Conceptual Framework

The conceptual framework was built from water balance and water losses concepts. The dependent variable represents the output or effect (nodal pressure levels), the independent variables represent the inputs or causes, or are tested to see if they are the cause. Moderating variables are undesirable variables that influence the relationship between the variables that an experimenter is examining. It usually bears an effect on the behavior of the subject being studied and it is assumed to influence the dependent variable. Figure 2.4 represents the conceptual framework for this study in which the independent variables are mapped to dependent variable and how extraneous factors come into play.

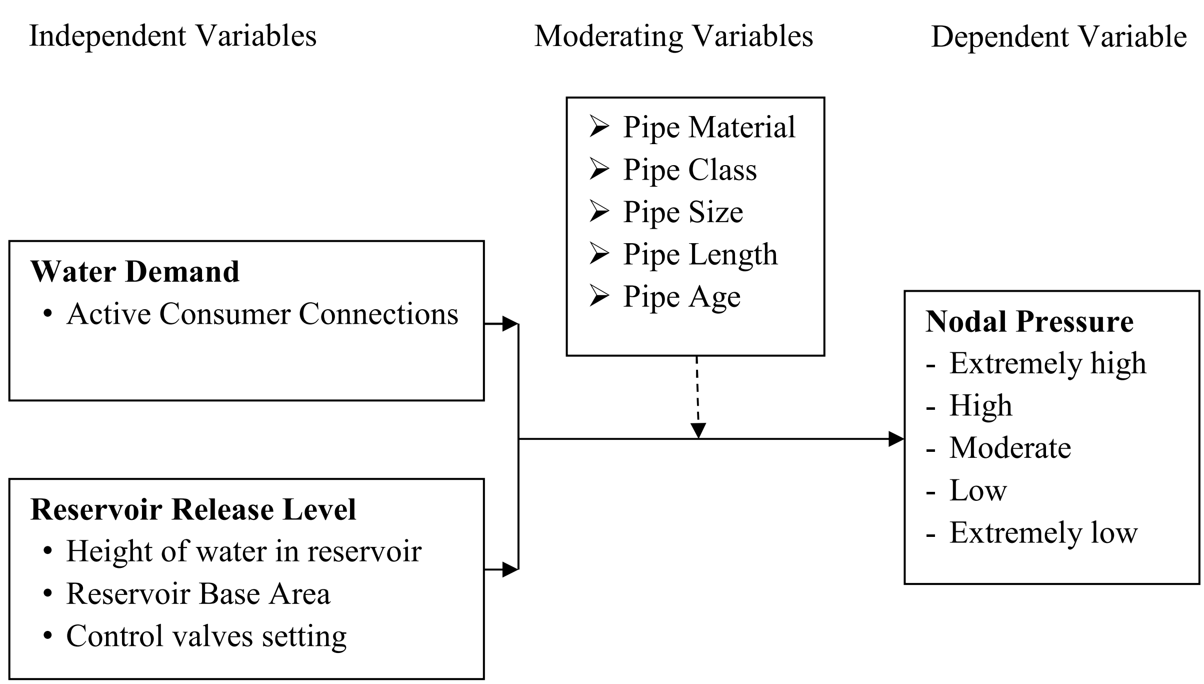



**Figure 2. 4: Conceptual Frame Work.**

In this conceptual framework the independent variables are the water supply system water demand and reservoir water release levels that do affect the variations in pipeline nodal pressures, the moderating variables are water supply system pipeline material, class, size, length and age, they influence the effect of water demand and reservoir water release levels on the pipeline nodal pressures. The dependent variable is the system nodal pressure that varies with change in system water demand and system reservoir water release levels.

# CHAPTER THREE: RESEARCH METHODOLOGY

## 3.1 Introduction

This chapter discusses the materials and methods used in this research. This chapter is divided into eight sections; study area, research design, target population, data and data sources, data collection instruments, data processing , data analysis and ethical considerations.

## 3.2 Study Area

This research was carried out at Kimilili water supply scheme area falling under the jurisdiction of Nzoia Water Services Company that operates in a cluster of five urban settings namely Kitale, Bungoma, Webuye, Kimilili and Chwele. Nzoia Water Services Company started operations in February 2005. The clustered company covers a combined coverage area of 405Km$^2$, 110km$^2$ in Trans-Nzoia County specifically Kitale town with its environs, 70km$^2$, 80km$^2$, 100km$^2$ and 45Km$^2$ in Bungoma, Webuye, Kimilili and Chwele towns and their environs respectively in Bungoma county (Figure 3.1).

The water coverage for Kimilili water supply scheme is 65%, the mean total precipitation of the area is 1400mm/year, relative humidity is between 65% and 63%, and the average temperature is 24.5 °C. The raw water for Kimilili water supply is abstracted from River Kibisi via intake works located about 7.5km from Kimilili town, then raw water is gravitated to the water treatment works located at Kamtiong'o through three 150mm, 3.2km long each parallel uPVC class 'D' pipelines. Kamtiong'o Water Treatment works is situated at the foot of Mt. Elgon (N00$^o$ 48' 56") (E34$^o$ 42' 10") and

1755m ASL 4.3Km from Kimilili town. Kimilili water supply scheme lies within 0° 47' 0" N, 34° 43' 0" E (UTM Northing: 86621.02 Easting: 691036.93 Zone: 36N). The total average water production per year is 736,248m$^3$ with a customer base of 4,948 connections. Kimilili water supply is a gravity scheme with a design capacity of 5000m$^3$/d and utilizes electricity for pumping back wash water. Treated Water is stored in a 2500m$^3$ ground reinforced concrete clear water reservoir then gravitated to Kimilili town via 250mm and 200mm uPVC parallel pipelines.The distribution network amounts to about 87 km in length, the pipes are a mixture of AC (1.8%), GI (10.2%) and uPVC (88%), (NZOWASCO, 2014/2015).

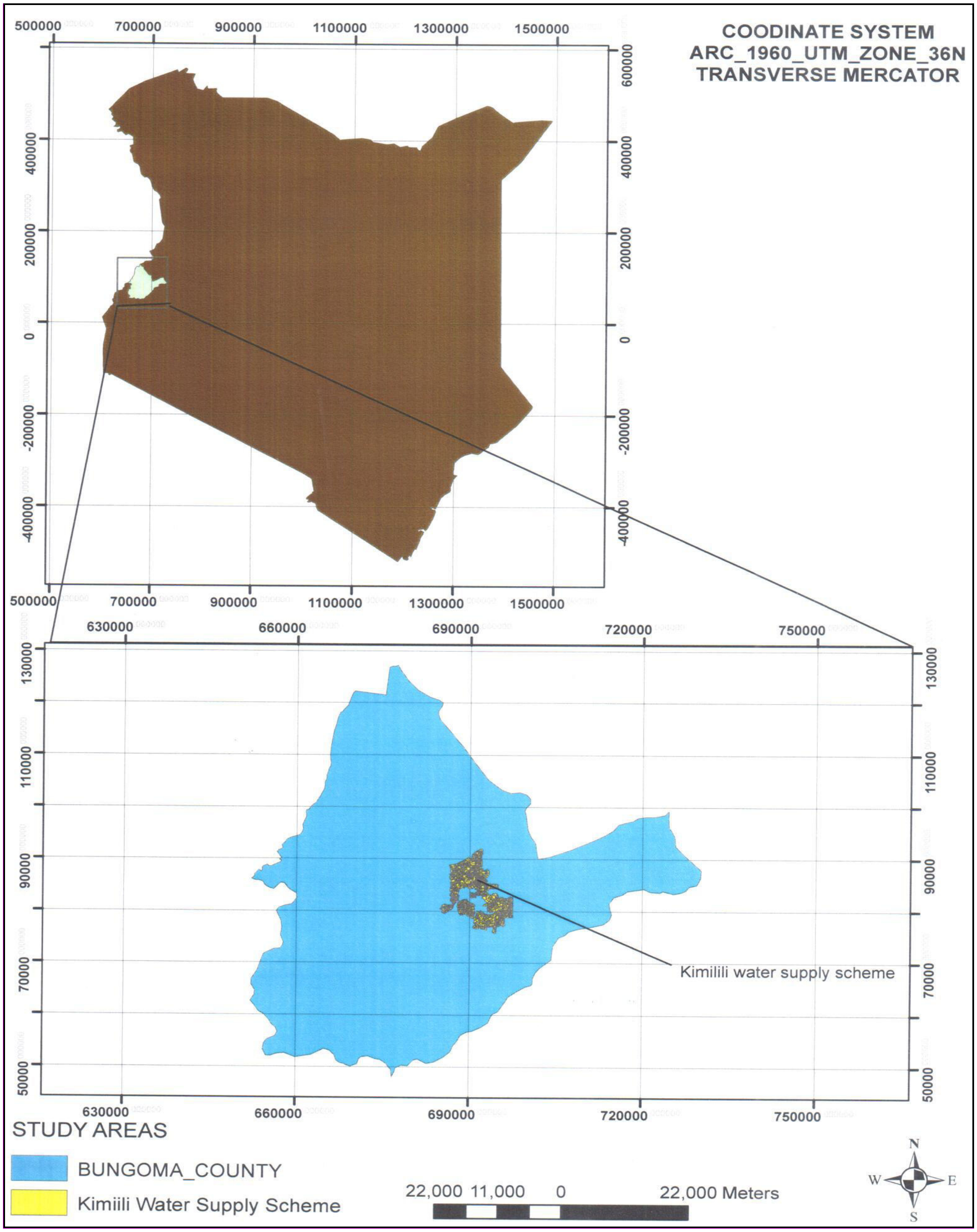



**Figure 3. 1: Kimilili Water Supply Scheme Coverage Area.**

Source: (NZOWASCO, 2015)

### 3.3 Research Design

The research design refers to the overall strategy that was chosen to integrate the different components of this study in a coherent and logical way, thereby, ensuring that the research problem was effectively addressed. This research adopted both historical and experimental designs. For historical design, historical water consumption data and water losses data were used to forecast the schemes' water demand while for the experimental design, actual zonal nodal pressures and flows were measured based on actual storage reservoir water release levels.

### 3.4 Target Population

The target population comprised of all individuals, objects or things that the researcher could reasonably generalize findings to. The study targeted all the varying active water consumer connections consumption trends over the years 2008 to 2016 mainly categorized into four consumer classes (domestic, commercial, institutional and communal) of Kimilili water supply scheme. The study also targeted the entire six zonal off take points (nodal) of Kimilili water supply distribution network. Table 3.1 shows population frame for time varying trends among domestic, commercial, institutional and communal water connections used in this research.

**Table 3. 1: Population Frame for Temporal Trends Among Domestic, Commercial, Institutional and Communal Connections.**

| | Category | 2008 | 2009 | 2010 | 2011 | 2012 | 2013 | 2014 | 2015 | 2016 |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Domestic | 1083 | 1146 | 1205 | 1293 | 1641 | 2135 | 2911 | 3391 | 3775 |
| 2 | Commercial | 694 | 712 | 768 | 797 | 823 | 874 | 934 | 985 | 1028 |
| 3 | Institutional | 46 | 48 | 51 | 57 | 63 | 81 | 86 | 91 | 94 |
| 4 | Communal | 24 | 28 | 27 | 29 | 29 | 38 | 43 | 49 | 51 |
| | **Total** | **1847** | **1934** | **2051** | **2176** | **2556** | **3128** | **3974** | **4516** | **4948** |

### 3.5 Data and Data Sources

Table 3.2 shows the summary of data and data sources used in this study. They are also described in the sections that follow.

**Table 3. 2: Data and data sources.**

| | Data type | Format | Source |
|---|---|---|---|
| 1. | Water connections | Monthly data in text file (Excel) | Nzoia Water Company |
| 2. | Water demand | Monthly data in text file (Excel) | Nzoia Water Company |
| 3. | Water losses | Monthly data in text file (Excel) | Nzoia Water Company |
| 4. | Pipeline network | GIS data in shape file | Nzoia Water Company |
| 5. | Nodal elevations | GIS data and observed data in text file | Field observations. |
| 6. | Nodal pressures | Observed daily data in text file (Excel) | Field observations. |

#### 3.5.1 Water Connections Data

This is the number of consumer connections supplied with potable water from Kimilili water supply scheme system. The connections were categorized into domestic connections, commercial connections, institutional connections and communal connections. This data was obtained from Kimilili water supply scheme billing system (Water and Sanitation Information System [WASANIS)]) data base for the period 2008 to 2016.

#### 3.5.2 Water Demand Data

This refers to the volume of water in cubic meters supplied to each of the consumer connections on monthly basis. The data was obtained from Kimilili WASANIS data base for the period 2008 to 2016.

### 3.5.3 Water Losses Data

Water losses refer to the amount of water in cubic meters that is injected into the distribution network but is not delivered to the consumers. It is usually obtained by subtracting the amount of water billed to all the consumers from amount of water injected into distribution network. The data was obtained from Kimilili water supply scheme monthly reports for the period 2008 to 2016.

### 3.5.4 Pipeline Network Data

This refers to the data that defines the water supply pipeline distribution framework including the hydraulic components. The attributes of the water supply distribution pipeline framework include; pipe length, pipe internal diameter, pipe roughness and pipe loss coefficient. Besides the pipeline, other hydraulic components include water reservoirs, break pressure tanks, pumps, valves and nodes just to mention a few. This data was obtained from Nzoia Water Services Company, Kimilili water supply scheme pipeline network GIS data base.

### 3.5.5 Nodal Elevation Data

This is data that defines the position of nodal points (pipeline junction joints) in relation to the height above sea level (meters or feet). The data was obtained from Nzoia Water Services Company Kimilili water supply pipeline network GIS maps and for confirmation purposes, physical measurements were taken from the field based on already established temporary bench mark (TBM) stations to ascertain the accuracy of the data.

### 3.5.6 Nodal Pressure Data

The data describes the variations in hydrostatic pressure distribution at selected points in the water supply pipeline distribution network. The points in mind were the various off take points (junctions) of the zonal pipelines from main distribution pipelines. This data was used to calibrate the EPANET simulation model and was directly obtained from the field using pressure data loggers installed in the pipeline network.

## 3.6 Data Collection Instruments

The core of the study was formed by both primary and secondary data that were collected from the field and from the company records respectively. Primary data was collected through field observations using GPS handset, pressure loggers and clamp on ultrasonic flow meter. For (x,y) coordinates data, GARMIN GPS handsets (GPSmap 60CSx and etrex 30x) were used to collect the data, TECHNOLOG Cello GSM Data Loggers and KELLER IM manometer were used to collect zonal nodal points pressure data while a clamp on Flexim Fluxus ADM 6725 ultrasonic flow meter was used to collect linkage (pipe) water flows. Secondary data was obtained through document review of pipeline network design data, system reports and management reports. Table 3.3 shows the data collection instruments used in the study.

**Table 3. 3: Data Collection Instruments**

| | Instrument | Application in the study | Functional Features |
|---|---|---|---|
| 1 | Garmin GPSmap 60CSx | ➢ Pipeline location<br>➢ Nodes location<br>➢ Appurtenances location<br>➢ Waypoint creation | ➢ Electronic compass<br>➢ Electronic altimeter<br>➢ Waypoint creator<br>➢ Route creator<br>➢ Profile creator<br>➢ Trip computer |
| 2 | Garmin etrex 30x | ✓ Pipeline location<br>✓ Nodes location<br>✓ Appurtenances location<br>✓ Waypoint creation | ✓ Electronic compass<br>✓ Electronic altimeter<br>✓ Waypoint creator<br>✓ Route creator<br>✓ Profile creator<br>✓ Track creator<br>✓ Trip computer |
| 3 | Technolog Cello GSM Data logger (PMAC Plus) | ➢ Fixed point pressure monitoring | ➢ Electronic pressure monitor<br>➢ Electronic flow monitor<br>➢ Electronic temperature monitor |
| 4 | Keller IM Manometer | - Mobile pressure monitoring | - Electronic pressure monitor |
| 5 | Flexim Fluxus ADM 6725 | ➢ Flow and Velocity monitoring | ➢ Volume / mass flow monitor<br>➢ Flow velocity monitor<br>➢ Heat flow rate monitor<br>➢ Volume, mass heat totalizer<br>➢ Calculation – average, sum, difference |

### 3.7 Data Processing

Data processing involved collecting and converting or manipulating raw data into format that was interpretable by the water demand forecasting model (MATLAB's ANN) and the hydraulic simulation model (EPANET 2.0).

### 3.7.1 Water Demand Forecasting Data

MATLAB software recognizes data in text file in excel format. For forecasting the water demand for Kimilili water supply scheme, the monthly data reports for 108 consecutive months (2008 - 2016) for water connections and respective water demand (water billed) per each consumer category were generated from the Kimilili WASANIS billing system and then exported into excel file then saved. The monthly water losses figures for the period 2008 - 2016 were obtained from the annual company reports and entered into their respective monthly columns in the saved exported water connections and water demand excel file already generated by the billing system as shown in Table 3.4. The updated excel file was saved in readiness for loading into the water demand forecast model (neural network tool of the MATLAB (R2014a), platform).

**Table 3. 4: Water Demand Data Ready For Exporting to MATLAB**

| Months | Apr-10 | May-10 | Jun-10 | Jul-10 | Aug-10 | Sep-10 | Oct-10 |
|---|---|---|---|---|---|---|---|
| Domestic | 560,160 | 561,600 | 564,480 | 567,840 | 570,720 | 571,680 | 574,560 |
| Commercial | 876,000 | 883,200 | 890,400 | 894,000 | 900,000 | 904,800 | 913,200 |
| Institutional | 1,200,000 | 1,200,000 | 1,200,000 | 1,200,000 | 1,200,000 | 1,224,000 | 1,224,00 |
| Communal | 67,200 | 64,800 | 64,800 | 64,800 | 64,800 | 64,800 | 64,800 |
| Base Demand ($m^3$/hr) | 135.2 | 135.5 | 136.0 | 136.3 | 136.8 | 138.3 | 138.8 |

### 3.7.2 Hydraulic Simulation Data

Preparation of data for simulating water supply distribution network nodal points pressure involved gathering two sets of data. The first set of data was for water demand which was generated from the ANN water demand forecast model. The forecasted water demand data generated by the ANN water demand forecast model in excel file was converted into monthly average water flows (liters per second) and saved in readiness for feeding into the EPANET 2.0 system for simulation. The second set of data was for development of the water supply pipeline distribution network. This set of data was obtained by exporting Kimilili water supply pipeline distribution network data from the ArcGIS 10.4 system to enhanced metafile (EMF) file and saving in readiness for loading in the EPANET 2.0 model simulation system. Figure 3.1 shows Kimilili water supply pipeline distribution network in GIS database ready for exporting to EPANET 2.0 in EMF format.

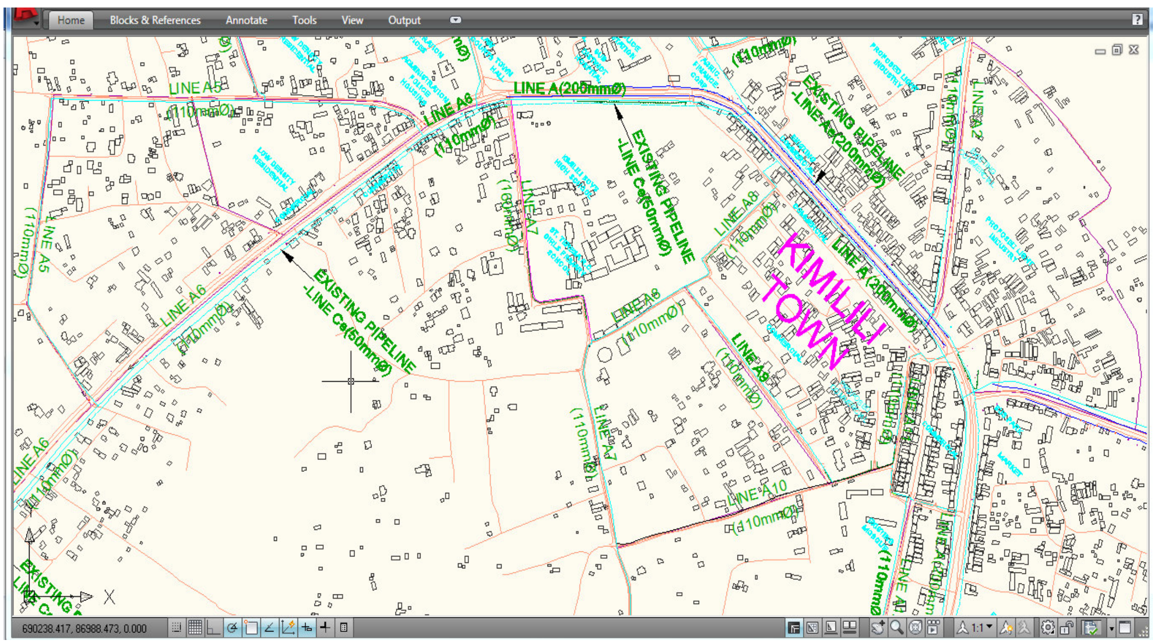



**Figure 3. 1: Kimilili Water Supply GIS System Pipeline Distribution Network**

### 3.8 Data Analysis

Data analysis entailed feeding the processed data into the water demand forecasting model (ANN) and the hydraulic simulation model (EPANET 2.0), running the models to produce output data and interpreting the output data into meaningful information.

### 3.8.1 Water Demand Forecasting.

The process of water demand forecasting for Kimilili water supply scheme involved four main steps;

a) Development of the Artificial Neural Network model for Water Demand Forecasting
b) Artificial Neural Network model calibration
c) Artificial Neural Network model validation
d) Water demand forecasting using Artificial Neural Network model.

**(i) ANN Model Development for Water Demand Forecasting**

ANN modeling tools are data based (black box) that works on the basis of deducing the future evolution demand from past tendencies, thus the modality of calculating water usage per category of users or application was not of great concern for the study. Development of the ANN water demand forecasting model involved;

i. Dividing the 108 months period water demand data set into three subsets as guided by (Smith, 1993) procedure for preparing and training an ANN; training subset (first 44 months – 40%), Validation subset (the second 32 months – 30%) and testing subset (last 32 months – 30%).
ii. Importing the data to the MATLAB workspace platform.
iii. In the MATLAB workspace platform, the variable data was further imported into the Neural Network/Data manager (nntool).
(iv) The next step entailed designing the neural network and deciding on the connection topology. The number of nodes in the input and output layers were

determined by the number of input and output variables under study. For this study the number of nodes in the input layer were six (five for the dependent variables – domestic connections, commercial connections, institutional connections, communal connections and water losses, and one for the independent variable – water demand). The output layer had one node (for the predicted water demand). Model calibration was the determinant of the number of hidden layers, the number of nodes in each hidden layer and the connection topology. Figure 3.2 shows the basic neural network for the water demand forecasting model, the dashed lines and nodes indicates that they were varying depending on the network design (the number of hidden layers and the number of nodes on each hidden layer) that produced the best performance (lowest validation error).

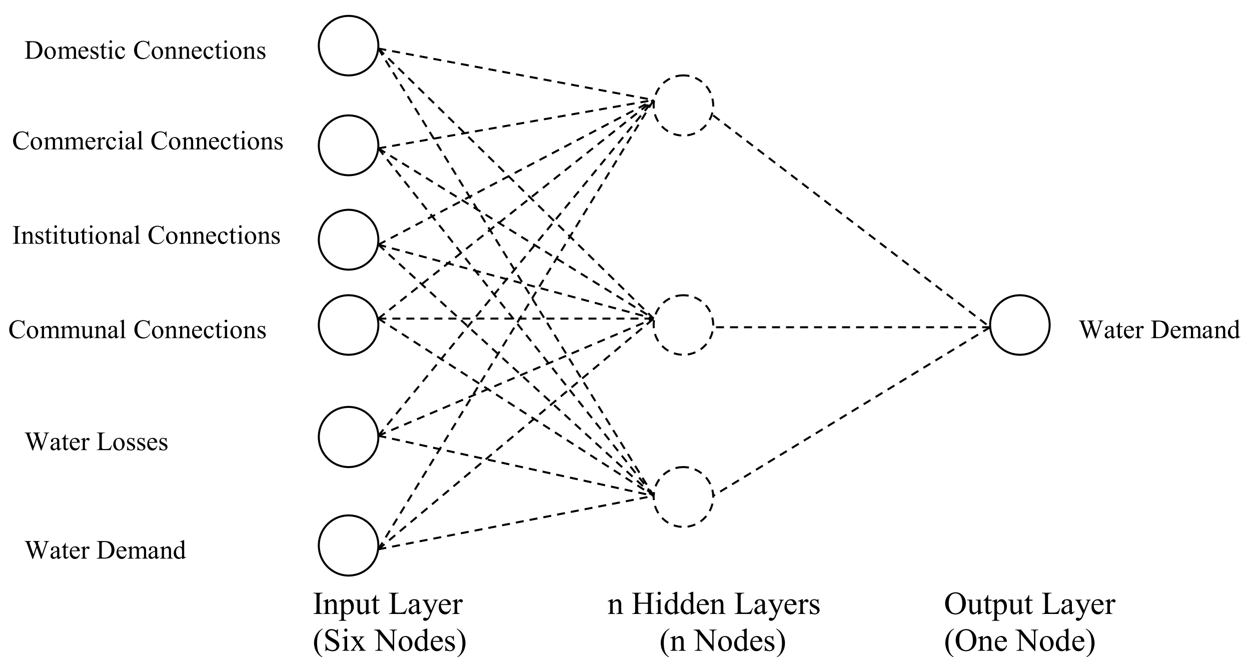



**Figure 3. 2: Basic Neural Network Model for Water Demand Forecasting**

During the network model development process, a total of twenty four network models were developed out of which network model 12 was adopted for the study as it produced the best results on testing. The network adopted was the layer recurrent type with TRAINLM (Levenberg Marquardt) training function, LEARNGDM (Gradient Descent with Momentum) learning function, MSE (Mean Square Error) performance function and TANSIG (Tan-Sigmoid) transfer function with three layers and six neurons.

Layer recurrent neural network is a dynamic network characterized by a feedback loop, with a single delay, around each layer of the network except for the last layer, it is trainable with gradient based algorithms. TRAINLM algorithm is the fastest training function and has the fastest convergence for networks that contain up to a few hundred weights and is able to obtain lower mean square errors than any of the other algorithms thus performs better on function approximation / fitting (nonlinear regression) problems. The hidden layer uses TANSIG as a transfer function.

**(ii) ANN Model Calibration**

The calibration process of the artificial neural network model being developed involved;

(i) Importing the training data subset into the MATLAB nntool.

(ii) Executing the system to train the network automatically based on the training data subset.

**(iii) ANN Model Validation**

The ANN model validation process involved;

(i) Periodically stopping of the model training and checking the performance error on validation data set.

(ii) The weights of the network were then saved.

(iii) Steps (ii) of the ANN Model calibration to steps (ii) of the ANN Model validation were repeated until when the error (MSE) on the validation data set started increasing (start of over fit).

(iv) The weights that produced the lowest error on the validation data set (step iii above) were then adopted for the trained ANN.

(v) The trained ANN was then tested using the testing data set to measure its performance (the predicted water demand was plotted along the 'observed' water demand and a correlation analysis was undertaken to assess the goodness of fit between the two data sets).

(vi) The network was redesigned and steps (ii) of the ANN Model calibration to steps (v) of the ANN Model validation were repeated until the model showed good performance.

**(iv) Kimilili water supply ANN model water demand forecasting**

The final process of forecasting Kimilili water supply water demand for the period 2017 to 2030 entailed;

(i) Importing of the saved excel file of water connections, water demand and water losses for the period 2008 to 2016 into the validated ANN model and updating it.

(ii) Running a simulation of the ANN model to generate water demand forecast results.

(iii) Saving of the generated results in an output folder in excel format.

### 3.8.2 Hydraulic simulation of Kimilili water supply distribution network nodal points pressure.

The process of creating EPANET 2.0 model for Kimilili water supply pipeline distribution network and simulating the nodal point pressures involved four major steps;

a) Development of EPANET 2.0 Model Network.

b) EPANET 2.0 Model Network Calibration.

c) EPANET 2.0 Model Network Validation.

d) Hydraulic simulation of Kimilili water supply distribution network model.

**(i) Development of EPANET 2.0 Distribution Network Model**

The process of development of Kimilili water supply EPANET distribution network model for nodal point's pressure simulation involved;

(i) Exporting Kimilili water supply distribution network data in the ArcGIS 10.4 system as shown in Figure 3.1 to enhanced metafile (EMF) file and saving the file.

(ii) A new project was opened in the EPANET 2.0 and default values were assigned. The default values assigned included; properties (pipe roughness - 140, auto length on) and hydraulics (flow units – LPS, head loss formula – H-W, specific gravity and Accuracy).

(iii) The network map was geo-referenced to map dimensions of; X,Y; 682547.765, 82919.984 and 699959.257, 90533.374 with map units in meters.

(iv) The saved EMF file was then loaded into the EPANET 2.0 platform as a backdrop layer for developing the water supply distribution network topology.

(v) Using EPANET system drawing tools, Kimilili water supply distribution network was developed with the various appurtenances incorporated.

(vi) The various components of the water distribution network were then assigned their attributes among them including pipe material, pipe diameter, pipe length, pipe roughness, junction elevation and valve type.

**(ii) EPANET 2.0 Distribution Network Model Calibration**

For the model calibration process, two groups of parameters were used. The first group composed of actual system data that included three main parameters that were obtained from the field (the water supply network distribution system), they were used for comparing the variance between the actual network hydraulic system data and the hydraulic simulation network model system input and output data. Actual water level in the tank was obtained using the tank water level gauge, actual link (pipe) flows were obtained from the field using a clamp on Flexim Fluxus ADM 6725 digital ultrasonic flow meter while actual network system pressures were obtained using Cello and Keller digital pressure data loggers.

The second group of parameters included the data that was feed into the EPANET 2.0 simulation system settings as system default values. This is data that was used to do the actual calibration of the simulation model system. This group consisted of three parameters; pipe roughness, head loss formula and emitter exponent. The procedure for the model calibration involved;

(i) Using actual data obtained from the field (system) at a given time, two calibration files one for the link flows and the other for nodal pressures were generated in notepad and saved as 'flow calibration' and 'pressure calibration' respectively.

(ii) Using actual data obtained from the field (system), the tank water level data and respective nodal water demands data at a given time were feed into the hydraulic simulation model.

(iii) The 'flow calibration' and 'pressure calibration' files were then imported into the EPANET hydraulic simulation platform as calibration files and saved one at a time.

(iv) The simulation system default value settings for pipe roughness, head loss formula and emitter exponent were then set. The discharge coefficients of the emitters and the settings of the flow control valves were updated using time varying demands in equation 2.17 ($q_j^{avl} = C_d\left(H_j^{avl} - H_j^{\min}\right)^{\Upsilon} : H_j^{avl} \geq H_j^{min}$).

(v) The hydraulic simulation model system was then run.

(vi) The calibration reports for correlations between means of observed and computed values for nodal point pressures and link flows where then generated separately from the hydraulic simulation system.

(vii) For correlation, a very strong relationship exists when $r$ = 0.80-1.0 (Douglas, 2003), thus steps (iv) to (vi) were repeated until when the correlation for link flows and nodal pressures was very strong leading to the adoption of the model. Figure 3.2 shows the analysis algorithm flow chart adopted for the EPANET hydraulic simulation model calibration process.

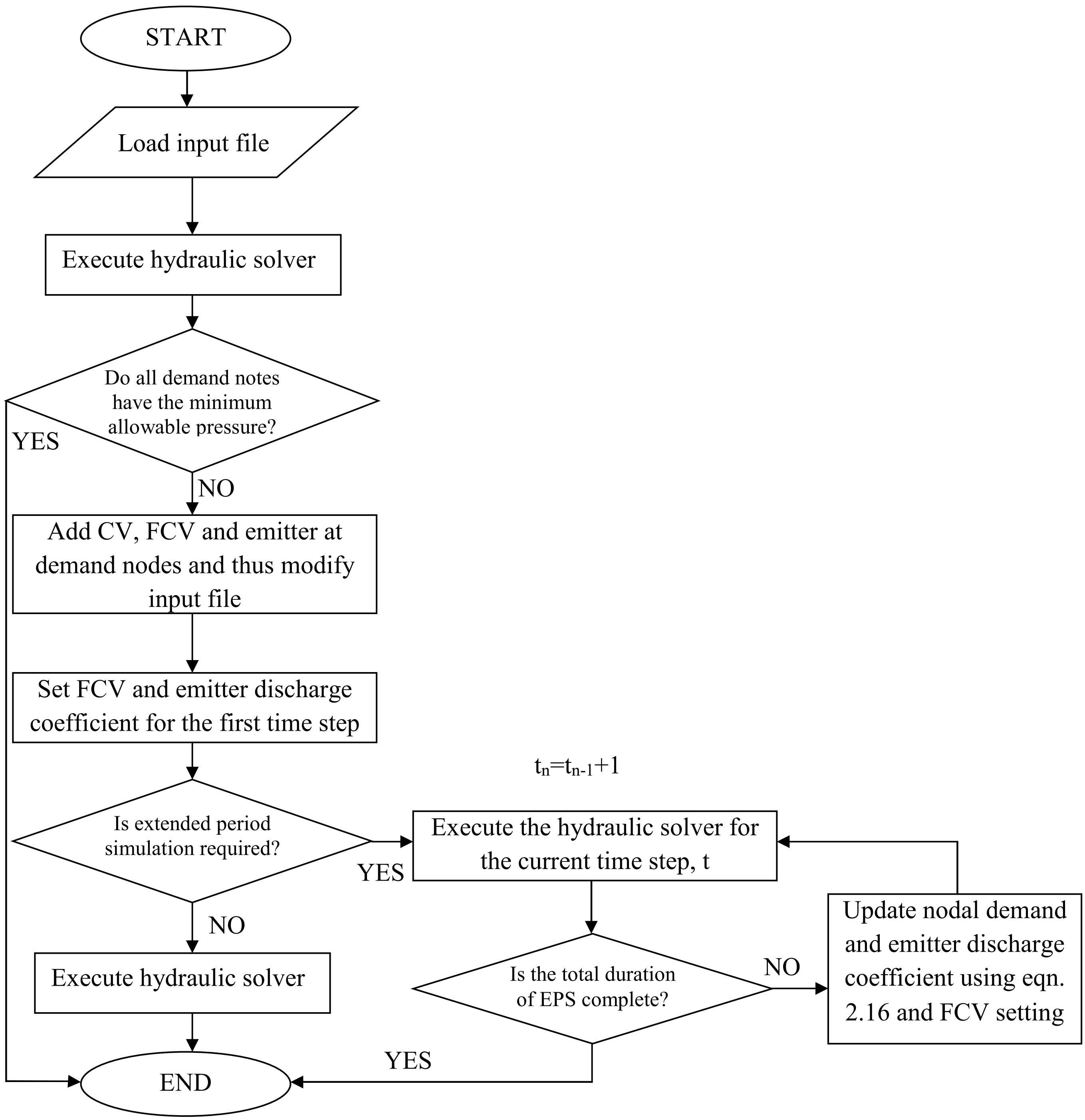



**Figure 3. 3: EPANET pressure dependent analysis algorithm flow chart.**

Source: Sayyed et al, 2015

### (iii) EPANET 2.0 Distribution Network Model Validation

Model validation involved the application of the calibrated model to a different set of nodal points and linkages without changing the parameters set in the calibration procedure. The process entailed;

(i) Both the tank water level and the immediate flow from the tank (network demand) were measured using tank water level gauge and clamp on ultrasonic flow meter respectively and then recorded.

(ii) At the same moment of measuring the tank water level and the immediate outflow, pressures of selected set of nodal points were measured using pressure data loggers and then recorded. At the same time, water flows for the immediate linkages (pipelines) feeding the nodal points were measured using clamp on ultrasonic flow meter and recorded.

(iii) Using the recorded pressure and flow data sets, two files, one for pressure validation and the other for flow validation were created in notepad and then saved.

(iv) The calibrated hydraulic simulation model was then fed with the recorded data for tank water level and network demand (flow).

(v) Using the EPANET 2.0 calibration tool, both the pressure validation and flow validation files were imported into the model.

(vi) The hydraulic simulation model system was then run.

(vii) The validation calibration reports for correlations between means of observed and computed values for nodal pressures and link flows where then generated separately from the hydraulic simulation system.

(viii) The validated distribution network was then saved.

**(iv) Hydraulic Simulation of Kimilili Water Supply Distribution Network Model.**

The hydraulic simulation of Kimilili water supply distribution network EPANET 2.0 model entailed;

(i) The ANN model forecasted and saved monthly water demand data for the period January 2017 to December 2020 was averaged on six months period basis (April to September being low water demand season and October to March being high water demand season), all forming nine water demand periods with the first and the last periods having three months each.

(ii) The periodic water demand data was then saved in excel format for further use.

(iii) The calibrated, validated and saved EPANET 2.0 Kimilili water supply distribution network model was opened.

(iv) The storage reservoir water release level was set at six meters while all the control valves were fixed at open status and updated in the opened EPANET 2.0 model.

(v) The water demand data for the first demand period (January 2017 to March 2017) was then fed into the EPANET 2.0 distribution network model then followed by running of the hydraulic simulation of the model.

(vi) A hydraulic simulation report for the first demand period was then generated using the EPANET 2.0 report link and saved in notepad format.

(vii) Steps (v) to (vi) were repeated for each of the remaining eight demand periods.

(viii) The results for the forecasted periodic water demand versus the system water losses for all the nine demand periods were then combined, presented in both tabular and graphical formats.

(ix) Further extended annual hydraulic simulation for the period 2021 to 2030 was carried out for ten consecutive periods each period being a calendar year through repetition of steps (v) to (viii).

### 3.8.3 Sensitivity Analysis of the model

Sensitivity analysis of the model was carried out to determine the possible policy and theoretical implications of the results, it was considered important to conduct sensitivity analysis. Sensitivity of the model in estimating system water losses was investigated through estimating; (i) periodic system water demand against system water losses, and (ii) periodic system water demand against percentage system water losses.

### 3.8.4 Establishment of optimal reservoir water release levels.

Establishment of the optimal reservoir water release levels entailed two stages, first determining the required nodal points pressure ranges and then determining the reservoir water release levels that would result to the generation of the required nodal point pressures at any given system water demand.

#### (i) Determination of required nodal point's pressures

This process involved determining Kimilili water supply scheme pipeline distribution network composition in terms of piping materials and their respective classes, after which the required nodal pressures were determined based on pressure rating guidelines for the pipeline material and class that formed the majority of the pipeline distribution network.

(i) The data on pipeline composition in terms of material and class of Kimilili water supply scheme pipeline distribution network was obtained from NZOWASCO 2014/15 annual report.

(ii) The percentage of the distribution network pipeline composition in terms of pipe material and class were then calculated, recorded and saved.

(iii) The majority composition of the distribution network pipeline was the established based on the percentage compositions.

(iv) International organization for standardization (ISO) pressure rating standard guidelines for the various pipe materials and the respective classes were then obtained.

(v) Based on the distribution network pipeline composition, the required nodal point's pressures were established from the ISO pressure rating standard guidelines for the pipeline material and class that formed the majority of the distribution network.

**(ii) Determination of the optimal reservoir water release levels.**

The process of determining the optimal reservoir water release levels of Kimilili water supply scheme pipeline distribution network for the various water demand periods involved;

(i) The monthly water demands generated by ANN water demand forecast model were averaged on six months period basis (April to September being low water demand season and October to March being high water demand season), all forming nine water demand periods with the first and the last periods having three months each.

(ii) The water demand data for each of the nine demand periods was then saved in excel format.

(iii) The water demand data for the first demand period (January 2017 to March 2017) was then feed and updated into the developed, calibrated and validated Kimilili water supply EPANET 2.0 model pipeline distribution network.

(iv) The storage reservoir water release level value was then set for the model starting with lower value and at the same time the control valves settings were adjusted.

(v) The model simulation was then run after which the generated zonal nodal pressure results were saved. The hydraulic simulation output reports generated were in tabular form.

(vi) Steps (iv) to (v) were repeated with iterative simulations being run for the first demand period data feed into the EPANET 2.0 model at various reservoir water release levels and control valves settings up to the point when the generated nodal point pressures for all the six zonal nodal points were between 3.8 bars to 5.1 bars.

(vii) The reservoir water release level at which all the six zonal nodal pressures lied within 3.8 bars to 5.1 bars range was then recorded and saved as the optimal reservoir water release level for the first demand period.

(viii) Steps (iii) to (vii) were repeated for each of the remaining eight demand period.

(ix) The results for the forecasted water demand versus the optimal reservoir water release levels for all the nine demand periods were then recorded and presented in both tabular and graphical formats.

(x) Further extended determination of annual optimal reservoir water release levels for the period 2021 to 2030 (10 periods) was carried out through repetition of steps (iii) to (ix).

### 3.9 Ethical Considerations

In the study, the researcher sought permission to carry out the research from the National Council of Science and Technology, and Nzoia Water Services Company Limited (Appendix I & II). NZOWASCO was assured of confidentiality of the information provided. Authorization for the research was also obtained from the county commissioner and the county director of education, Bungoma County. The source for each information was indicated and the researcher did not take any short cuts in obtaining data. The researcher further subjected the thesis report to plagiarism index check on Turnitin website of which a similarity report was generated which showed an overall similarity index of 8% (Appendix III & IV).

# CHAPTER FOUR: RESULTS AND DISCUSSION

## 4.1 Introduction

This chapter presents results of data analysis for the three objectives under study. It begins by giving the results of each process done and the discussions for the same results. Graphs and tables from MATLAB (R2014a) and EPANET 2.0 output have been incorporated for elaboration purposes. The first objective discussed in this chapter was to forecast Kimilili water supply scheme water demand up to 2030. The study focused on all active water connections and all the four consumer categories for prediction of water demand varying trends over years.

Prediction of the water demand was done on monthly basis while the analysis was done based on six month average period. The second objective discussion focused on simulation of Kimilili water supply distribution network zonal nodal pressures. The simulation was carried based on each six month period average water demand forecasted. The third objective was to establish optimal reservoir water release levels for maintaining minimum allowable zonal nodal pressures. The optimal reservoir water release levels were determined for each simulated Kimilili water supply distribution network zonal nodal pressures for each six month average water demand period.

## 4.2 Kimilili water supply water demand forecast

The forecasting process here involved development of the Artificial Neural Network model for water demand forecasting, Artificial Neural Network model calibration using the training data subset of 44 consecutive calendar months (Jan 2008 – Aug 2011), Artificial Neural Network model validation using the validation data subset of 32 consecutive calendar months (Sep 2011 – April 2014), Artificial Neural Network model testing using testing data subset of 32 consecutive calendar months (May 2014 – Dec 2016) and the forecasting of water demand of Kimilili water supply scheme for a total of forty eight consecutive months (four calendar years starting from January 2017 to December 2020) then extended annual water demand forecast from 2021 to 2030 using the developed ANN model.

### 4.2.1 ANN Model Development for Water Demand Forecasting

During the network model development process, a total of twenty four network models were developed out of which network model number 12 was adopted for the study as it produced the best results on testing. The network adopted was the layer recurrent type with TRAINLM (Levenberg Marquardt) training function, LEARNGDM (Gradient Descent with Momentum) learning function, MSE (Mean Square Error) performance function and TANSIG (Tan-Sigmoid) transfer function with three layers and six neurons.

### 4.2.2 ANN model calibration

The ANN model calibration process involved importation of the training data subset into the MATLAB nntool and training the network. The best calibration was attained with a performance of 4.69 at a gradient of 4.2955e-0.005 at epoch 291 with 341 iterations.

### 4.2.3 ANN Model Validation

The ANN model validation process entailed redesigning of the network by changing the connection topology, changing the weights of the network, training the model and periodically stopping of the model training, checking the performance error on validation data set and testing of the trained network. The best validation performance achieved was 0.00023942 at epoch 291. Figure 4.1 shows the results of the best validation performance attained as generated by the system during the network model validation.

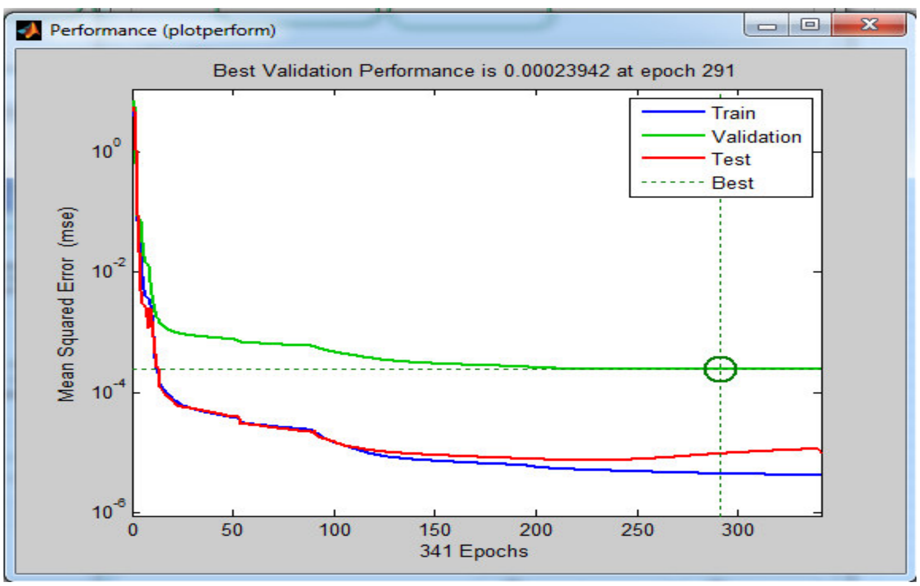



**Figure 4. 1: Neural Network M12 Best Validation Performance.**

The performance of the trained ANN was tested by carrying out correlation analysis between the predicted water demand and the observed water demand data sets using the correlation function of the Microsoft Excel.

A correlation coefficient (*r*) of 0.999972988 was attained. According to (Douglas, 2003) when *r* = 0.00-0.19 (very weak relationship), *r* = 0.20-0.39 (weak relationship), *r* = 0.40-0.59 (moderate relationship), *r* = 0.60-0.79 (strong relationship) and *r* = 0.80-1.0 (very strong relationship). Thus an *r* value of 0.999972988 was reasonable for this study, hence the model was adopted for this research.

Table 4.1 shows both the observed water demand obtained from Kimilili WASANIS data base and the predicted water demand for 32 consecutive months period (May 2014 – December 2016) while Figures 4.2 shows a plot of the 'observed' monthly water demand along predicted water demand for the 32 months period and the correlation between the 'observed' water demand and network M12 predicted water demand results respectively.

**Table 4. 1: Observed Water Demand and Predicted Water Demand For 32 Months.**

| Months | Observed Water Demand ($m^3$/h) | Network M12 Predicted Water Demand ($m^3$/h) | Months | Observed Water Demand ($m^3$/h) | Network M12 Predicted Water Demand ($m^3$/h) |
|---|---|---|---|---|---|
| May-14 | 96.36 | 96.37 | Sep-15 | 98.36 | 98.37 |
| Jun-14 | 96.53 | 96.53 | Oct-15 | 98.53 | 98.54 |
| Jul-14 | 96.69 | 96.70 | Nov-15 | 98.70 | 98.70 |
| Aug-14 | 96.83 | 96.84 | Dec-15 | 98.87 | 98.90 |
| Sep-14 | 96.96 | 96.97 | Jan-16 | 99.08 | 99.09 |
| Oct-14 | 97.09 | 97.10 | Feb-16 | 99.30 | 99.31 |
| Nov-14 | 97.23 | 97.23 | Mar-16 | 99.52 | 99.53 |
| Dec-14 | 97.36 | 97.37 | Apr-16 | 99.74 | 99.75 |
| Jan-15 | 97.52 | 97.53 | May-16 | 99.96 | 99.98 |
| Feb-15 | 97.69 | 97.65 | Jun-16 | 100.19 | 100.20 |
| Mar-15 | 97.76 | 97.77 | Jul-16 | 100.41 | 100.42 |
| Apr-15 | 97.84 | 97.84 | Aug-16 | 100.64 | 100.65 |
| May-15 | 97.91 | 97.91 | Sep-16 | 100.87 | 100.88 |
| Jun-15 | 97.98 | 97.99 | Oct-16 | 101.09 | 101.11 |
| Jul-15 | 98.06 | 98.10 | Nov-16 | 101.32 | 101.34 |
| Aug-15 | 98.21 | 98.21 | Dec-16 | 101.56 | 101.57 |

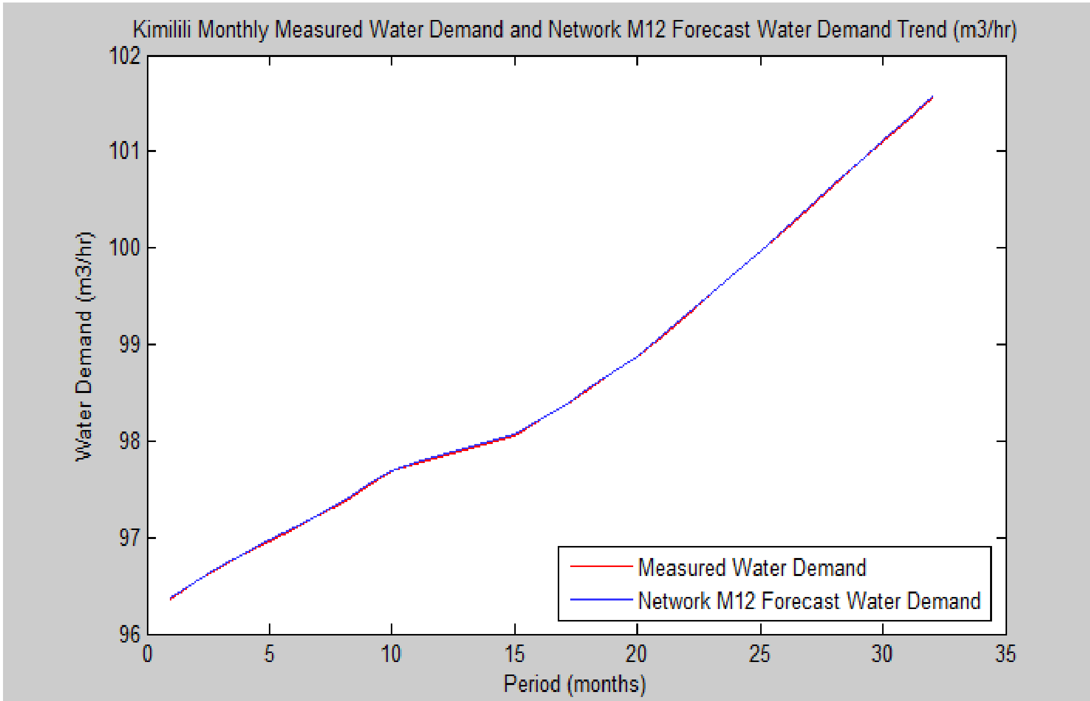



**Figure 4. 2: A plot of Network M12 32 months period monthly 'observed' water demand along predicted water demand.**

### 4.2.4 Kimilili water supply ANN model water demand forecast

Neural network tool of MATLAB R2014a of the developed, calibrated, validated and tested Kimilili water supply ANN model was used to forecast water demand for Kimilili water supply scheme for a total of forty eight consecutive months (four calendar years starting from January 2017 to December 2020). The results for the forty eight months forecasted water demand data are as shown in table 4.2 and Figure 4.3.

**Table 4. 2: Kimilili Water Supply Water Demand Forecast (Jan 2017 – Dec 2020).**

| Months | Demand ($m^3$/h) | Months | Demand ($m^3$/h) | Months | Demand ($m^3$/h) | Months | Demand ($m^3$/h) |
|---|---|---|---|---|---|---|---|
| Jan-17 | 100.19 | Jan-18 | 108.11 | Jan-19 | 118.32 | Jan-20 | 126.39 |
| Feb-17 | 101.42 | Feb-18 | 109.02 | Feb-19 | 119.42 | Feb-20 | 126.75 |
| Mar-17 | 102.50 | Mar-18 | 110.30 | Mar-19 | 119.86 | Mar-20 | 126.76 |
| Apr-17 | 102.53 | Apr-18 | 110.44 | Apr-19 | 120.40 | Apr-20 | 126.32 |
| May-17 | 102.44 | May-18 | 110.87 | May-19 | 121.20 | May-20 | 125.83 |
| Jun-17 | 103.24 | Jun-18 | 111.45 | Jun-19 | 121.90 | Jun-20 | 125.87 |
| Jul-17 | 104.44 | Jul-18 | 112.30 | Jul-19 | 121.98 | Jul-20 | 126.01 |
| Aug-17 | 104.90 | Aug-18 | 113.50 | Aug-19 | 122.50 | Aug-20 | 126.17 |
| Sep-17 | 105.65 | Sep-18 | 114.60 | Sep-19 | 123.60 | Sep-20 | 126.50 |
| Oct-17 | 107.18 | Oct-18 | 115.60 | Oct-19 | 124.40 | Oct-20 | 127.77 |
| Nov-17 | 107.41 | Nov-18 | 116.40 | Nov-19 | 125.60 | Nov-20 | 129.34 |
| Dec-17 | 107.91 | Dec-18 | 117.50 | Dec-19 | 126.20 | Dec-20 | 130.55 |

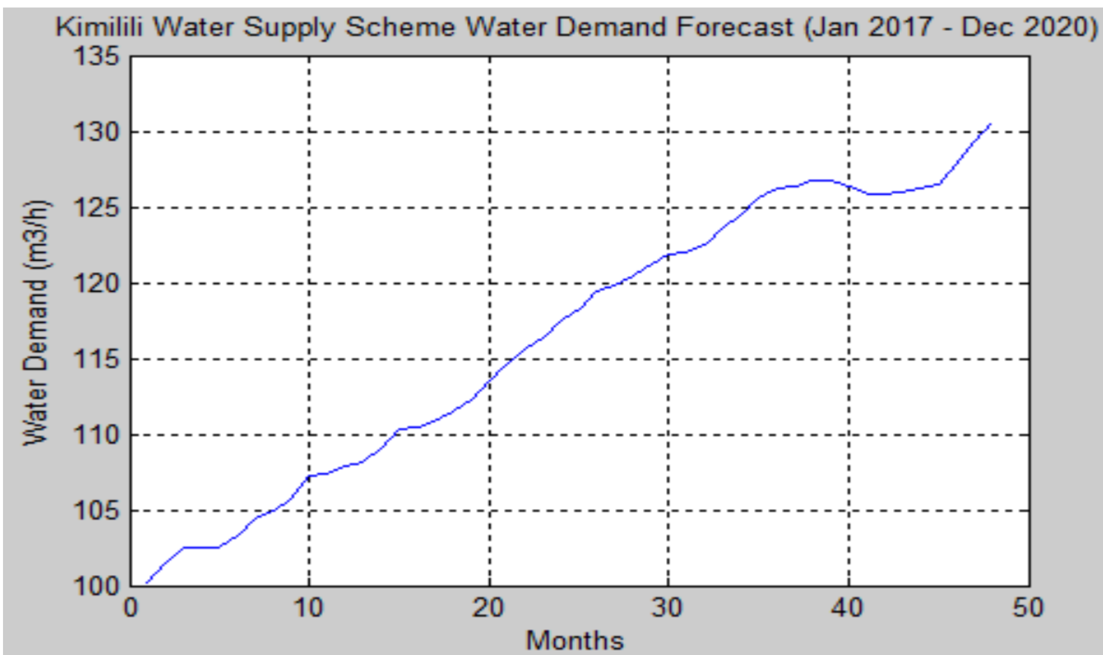



**Figure 4. 3: Kimilili Water Supply Water Demand Forecast (Jan 2017 – Dec 2018).**

The forecasted water demand trend indicates that by January 2017 the water demand for Kimilili water supply was 100.19$m^3$/h (27.83 l/s) translating to 2,405$m^{3/}$d while at the end of the forecasted period (December 2020) the water demand will be 130.55$m^3$/h (36.26 l/s), translating to a daily water demand of 3,133$m^3$. On yearly basis the water demand trend indicates that between October and March the demand for water is generally high and between April and September the demand is generally low. This is attributed to the fact that from October to March the rains are usually light (low) thus the consumers tend to depend mostly on tap water supplied from Kimilili system resulting to high demand. From April to September the rains are generally high leading to the consumers having an alternative source of water thus depending less on the tap water from Kimilili system consequently leading to low water demand.

From the forecasted water demand curve, the best curve of fit was drawn using excel for the relationship between the water demand and period, it was established that the general relationship between water demand and period is a polynomial function of order six defined by Equation 4.1 and figure 4.4.

$y = 9e\text{-}0x^6 - 1e\text{-}05x^5 + 0.0005x^4 - 0.0115x^3 + 0.1178x^2 + 0.1384x + 100.48$ …... Equation (4.1)

Where;

y = water demand in $m^3/hr$

x = period in months

e = standard error value of the coefficients.

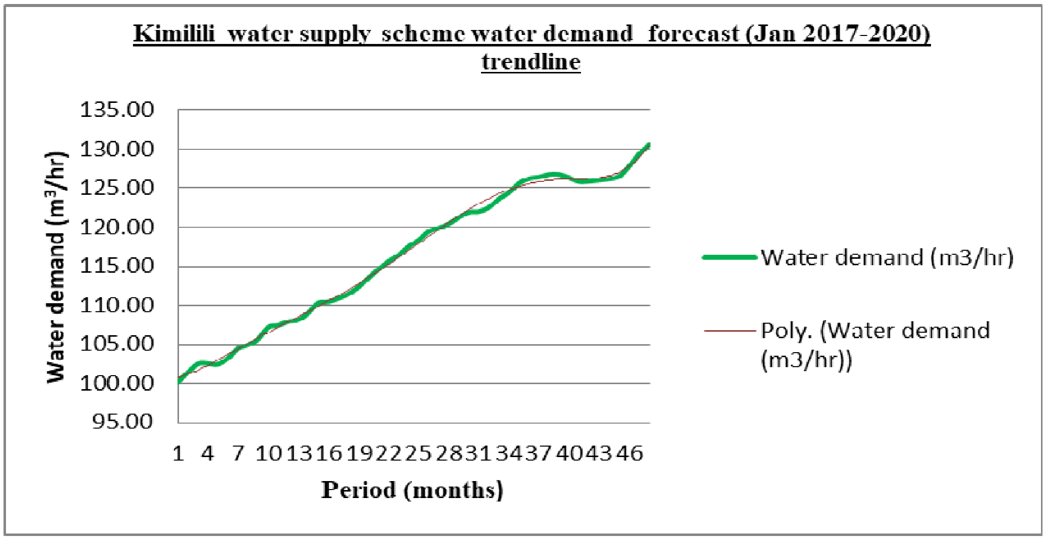



**Figure 4. 4: Kimilili Water Supply Scheme Water Demand Forecast Trendline.**

For hydraulic simulation purpose, the averaged three month / six months period water demand generated was 104.83m$^3$/hr for January to March 2017 and 132.55m$^3$/hr for the period October 2020 to December 2020. Table 4.3 and Figure 4.5 shows the averaged three months or six months period water demand for adoption for the hydraulic simulation.

**Table 4. 3: Kimilili Water Supply Scheme Forecasted Averaged Three / Six months Water Demand.**

| Period | Averaged water demand (m$^3$/hr) | Averaged water demand (l/s) |
|---|---|---|
| Jan 2017 - March 2017 | 104.83 | 29.12 |
| April 2017 - Sep 2017 | 107.32 | 29.81 |
| Oct 2017 - March 2018 | 111.74 | 31.04 |
| April 2018 - Sep 2018 | 115.60 | 32.11 |
| Oct 2018 - March 2019 | 121.25 | 33.68 |
| April 2019 - Sep 2019 | 125.32 | 34.81 |
| Oct 2019 - March 2020 | 129.35 | 35.93 |
| April 2020 - Sep 2020 | 129.49 | 35.97 |
| Oct 2020 - Dec 2020 | 132.55 | 36.82 |

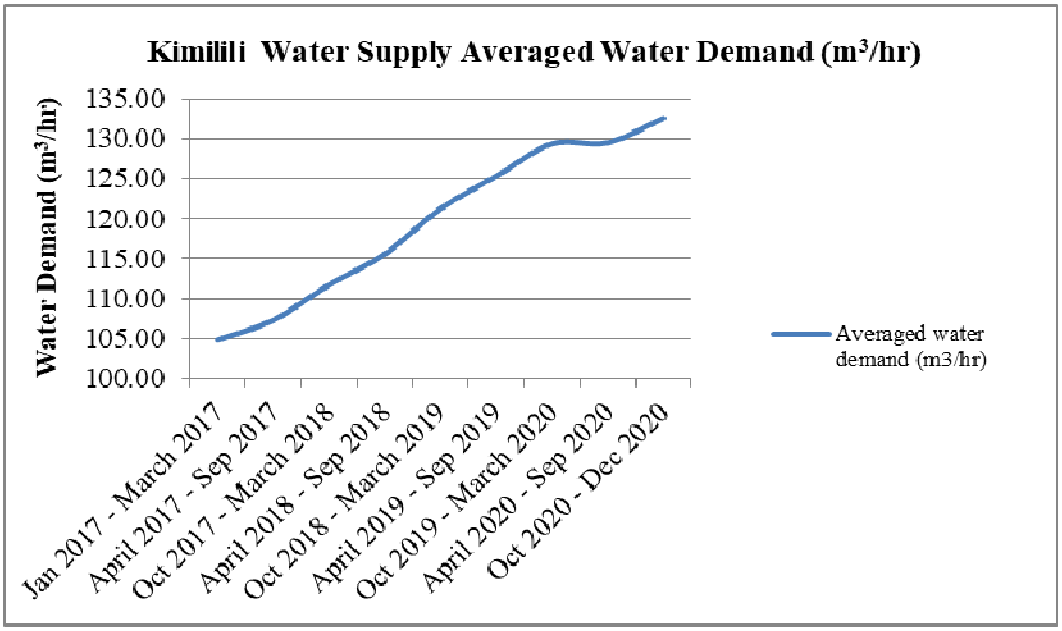



**Figure 4. 5: Kimilili Water Supply Scheme Forecasted Averaged three / four months Water Demand.**

Further extended annual water demand for Kimilili water supply scheme for a total of ten consecutive years (2021 to 2030) was forecasted using neural network tool of MATLAB R2014a of the developed, calibrated, validated and tested Kimilili water supply ANN model. The results for the ten years forecasted water demand data are as shown in Table 4.4 and Figure 4.6.

**Table 4. 4: Kimilili Water Supply Scheme Extended Annual Water Demand Forecast (2021 – 2030)**

| Year | Demand ($m^3$/h) | Year | Demand ($m^3$/h) |
|---|---|---|---|
| 2021 | 138.20 | 2026 | 183.56 |
| 2022 | 147.10 | 2027 | 200.09 |
| 2023 | 159.30 | 2028 | 213.48 |
| 2024 | 166.18 | 2029 | 222.55 |
| 2025 | 172.66 | 2030 | 228.38 |

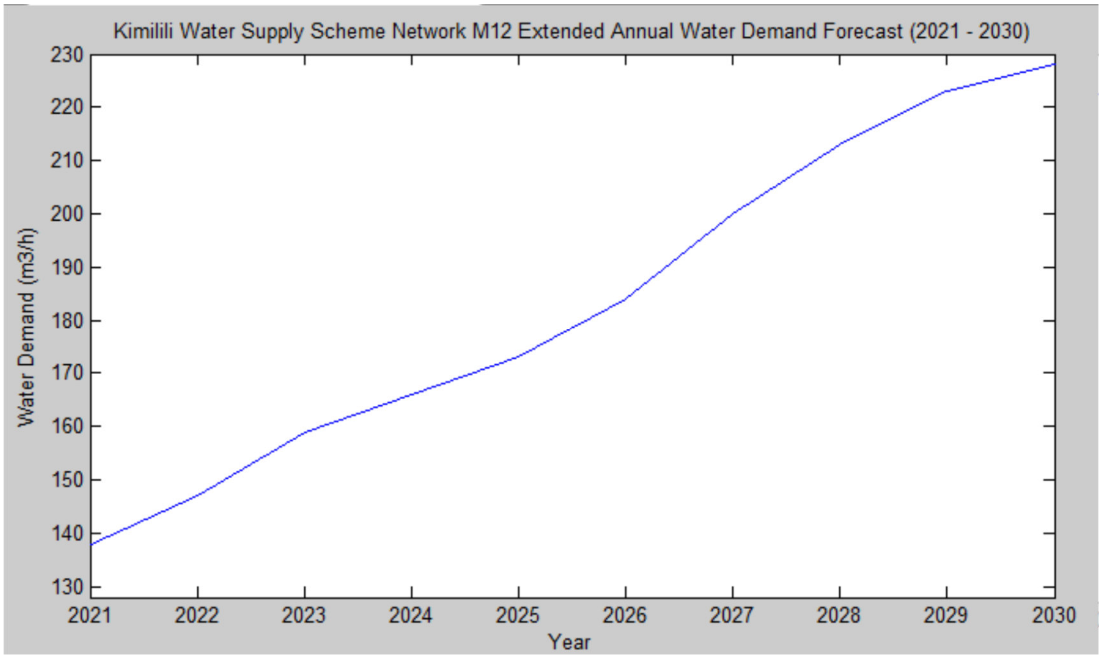



**Figure 4. 6: Kimilili Water Supply Scheme Extended Annual Water Demand Forecast (2021 – 2030).**

The forecasted extended annual water demand trend indicates that by 2021 the average annual water demand for Kimilili water supply scheme will be 138.20m$^3$/h (38.39 l/s) translating to 3,316.90m$^3$/d while at the end of the forecasted period (2030) the water demand will be 228.38m$^3$/h (63.44 l/s), translating to a daily water demand of 5,481.22m$^3$.

From the forecasted extended annual water demand curve, the best curve of fit was drawn using the basic fitting tool of MATLAB R2014a plot command window in order to establish the relationship between the annual water demand and period (year). It was established that the general relationship between annual water demand and period is a polynomial function of order five defined by Equation 4.2 and Figure 4.7.

$y = -0.0021x^5+22x^4-8.7e+0.4x^3+1.8e+0.8x^2-1.8e+11x+7.2e+13$ …..….. Equation (4.2)

Where;

y = water demand in $m^3/hr$

x = period in year

e = standard error value of the coefficients.

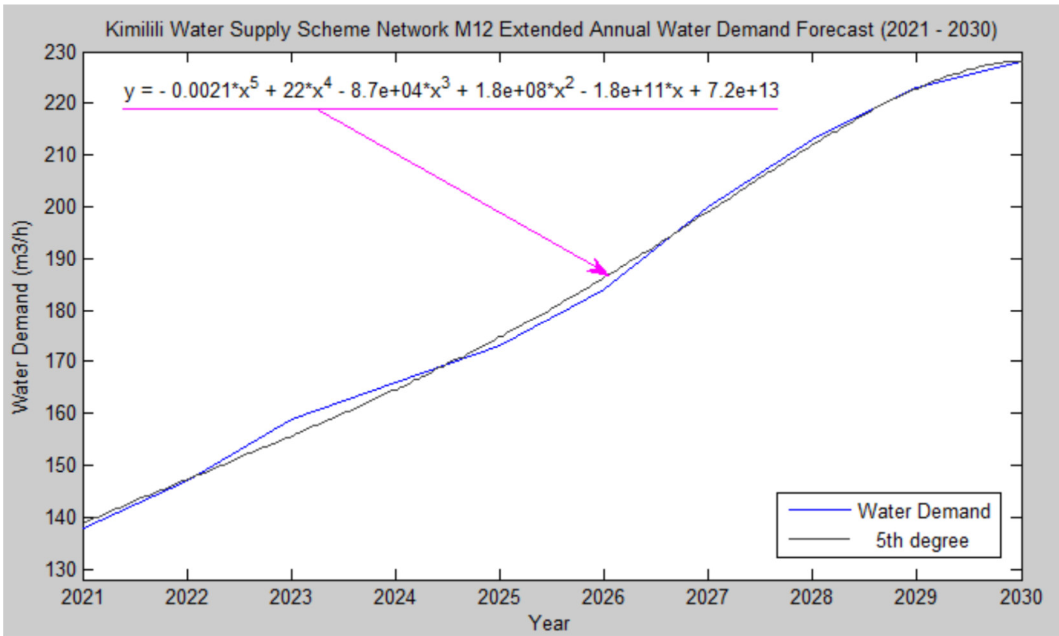



**Figure 4. 7: Kimilili Water Supply Scheme Extended Annual Water Demand Basic Fitting Trendline.**

### 4.3 Hydraulic simulation of Kimilili water supply distribution network nodal point's pressure.

The hydraulic simulation process entailed development of EPANET 2.0 Model network for Kimilili water supply scheme, calibration of the model network, validation of the model network and finally simulating the distribution model network nodal point's pressure.

### 4.3.1 Development of Kimilili water supply EPANET 2.0 Model Network.

The results for the developed Kimilili water supply scheme EPANET 2.0 model network are as presented in Figures 4.8, 4.9 and 4.10. Figure 4.8 shows the backdrop layer of Kimilili water supply EPANET 2.0 distribution network while Figures 4.9 and 4.10 shows the developed Kimilili Water Supply EPANET 2.0 mains distribution network with backdrop layer visible and backdrop layer hidden respectively.

Figure 4.8 shows the backdrop layer of Kimilili water supply Epanet 2.0 distribution network, it shows the land demacations, roads, rivers and building locations.

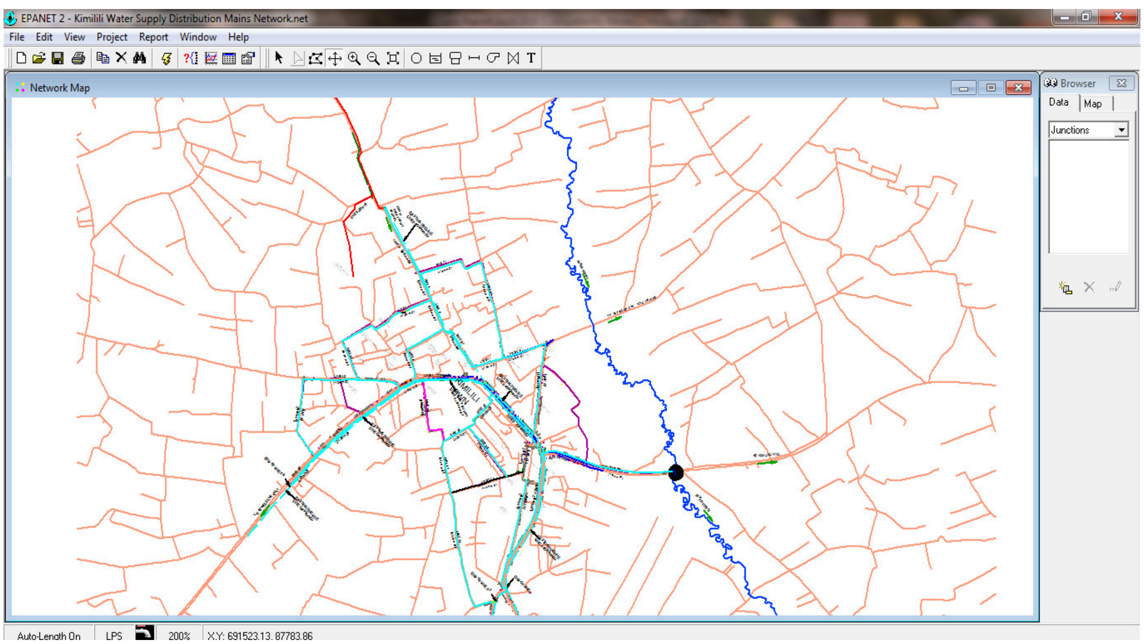



**Figure 4. 8: Kimilili Water Supply EPANET 2.0 backdrop layer of distribution network.**

Figure 4.9 shows Kimilili water supply Epanet 2.0 pipeline distribution network model with visible backdrop layer of the the distribution network. it shows the water reservoir, pipeline network, pipeline nodes, land demacations, roads, rivers and building locations.

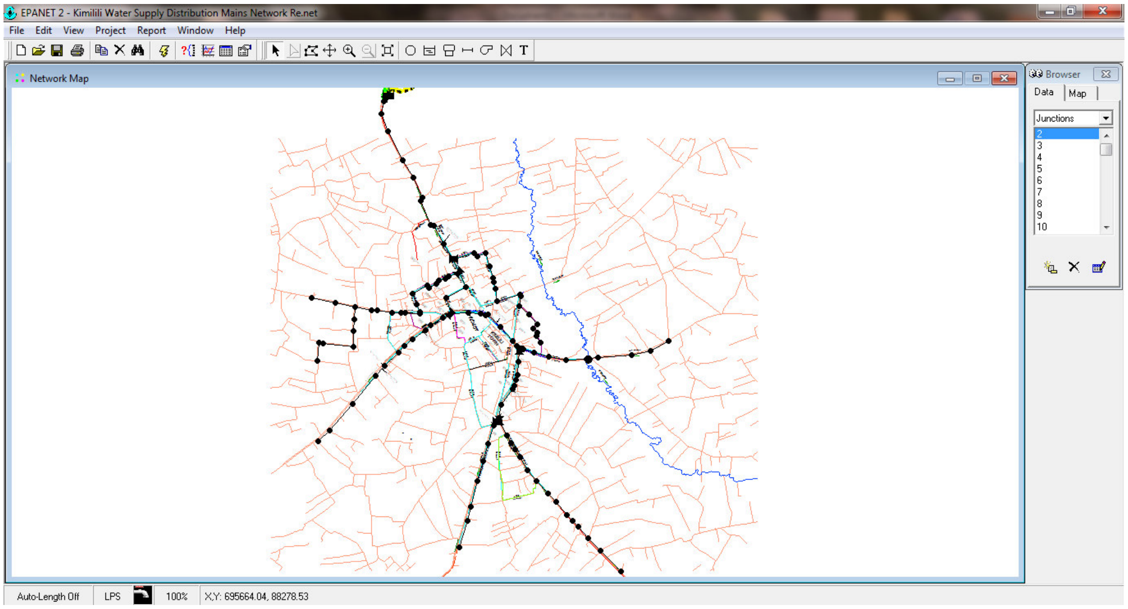



**Figure 4. 9: Kimilili Water Supply EPANET 2.0 pipeline distribution network model with visible backdrop layer of distribution network.**

Figure 4.10 shows Kimilili water supply Epanet 2.0 pipeline distribution network model with hiden backdrop layer of the the distribution network. It shows the water reservoir, pipeline network and the pipeline nodes.

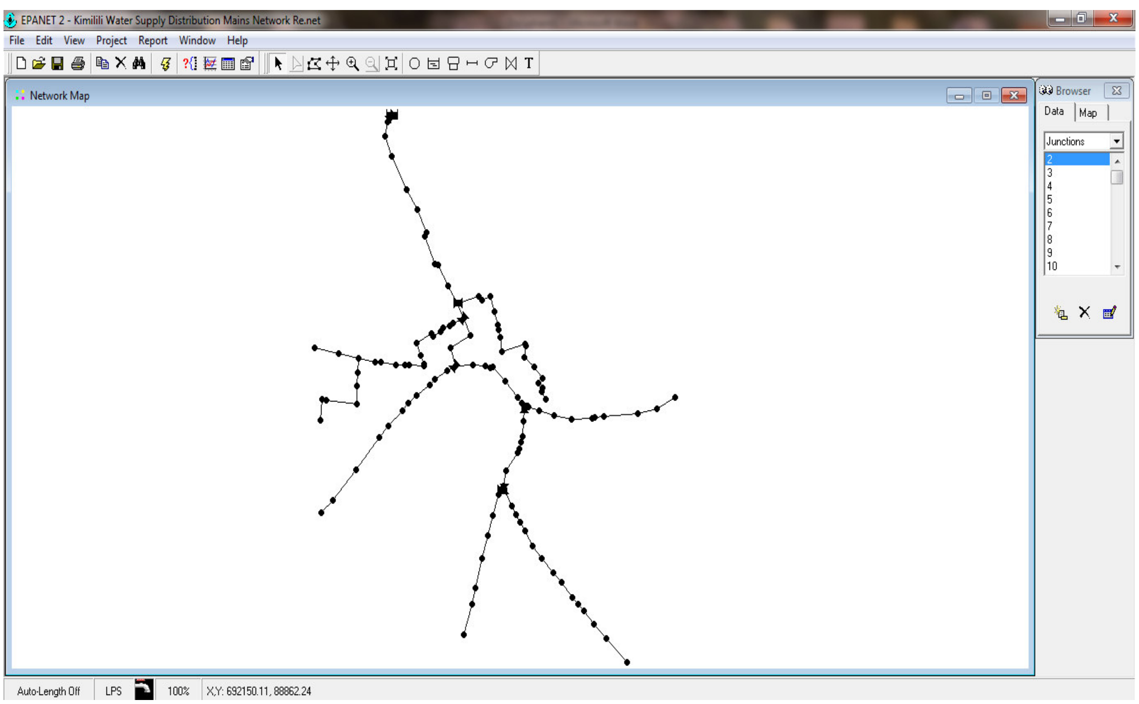
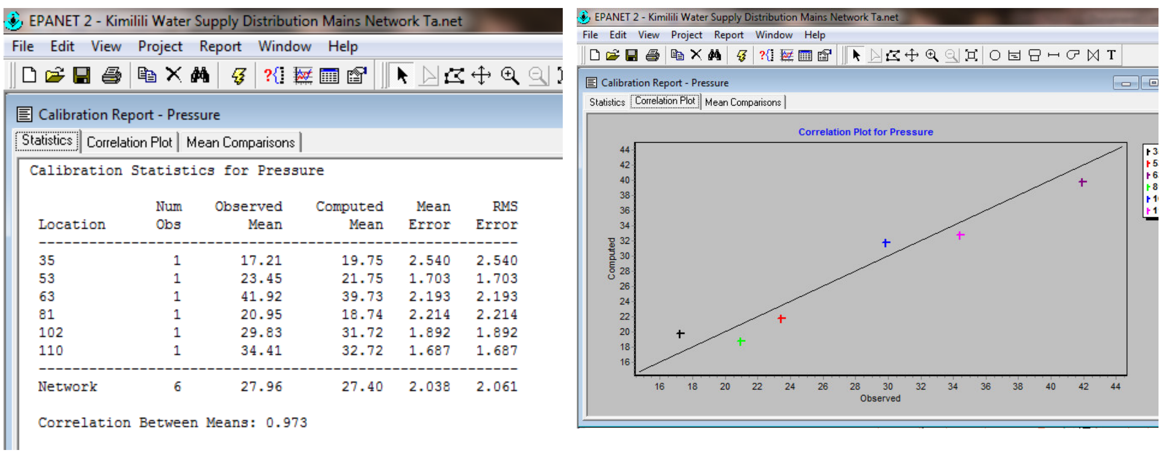



**Figure 4. 10: Kimilili Water Supply EPANET 2.0 pipeline distribution network model with hidden backdrop layer of distribution network.**

### 4.3.2 Kimilili Water Supply EPANET 2.0 Model Network Calibration.

For this research, successful model calibration was achieved when $r$ = 0.973 and $r$ = 0.936 for nodal point pressures and link flows respectively with EPANET setting default values of; 140, H-W, and 0.5 for pipe roughness, head loss formula and emitter exponent respectively. Figures 4.11 and 4.12 represent the final calibration reports for nodal point pressures and link flows respectively for the adopted model.

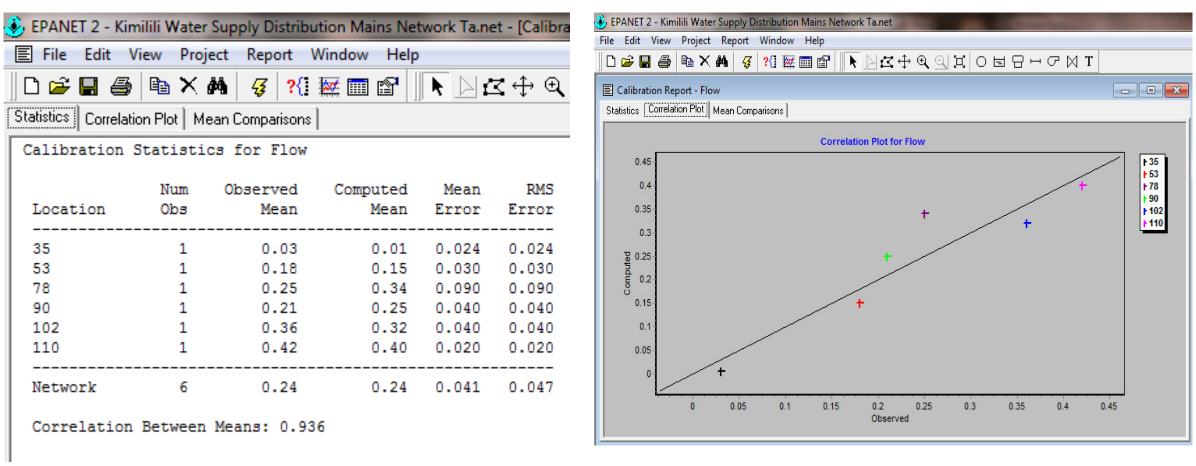



Calibration Statistics for Pressure

| Location | Num Obs | Observed Mean | Computed Mean | Mean Error | RMS Error |
|---|---|---|---|---|---|
| 35 | 1 | 17.21 | 19.75 | 2.540 | 2.540 |
| 53 | 1 | 23.45 | 21.75 | 1.703 | 1.703 |
| 63 | 1 | 41.92 | 39.73 | 2.193 | 2.193 |
| 81 | 1 | 20.95 | 18.74 | 2.214 | 2.214 |
| 102 | 1 | 29.83 | 31.72 | 1.892 | 1.892 |
| 110 | 1 | 34.41 | 32.72 | 1.687 | 1.687 |
| Network | 6 | 27.96 | 27.40 | 2.038 | 2.061 |

Correlation Between Means: 0.973

**Figure 4. 11: Nodal Points Calibration.**




Calibration Statistics for Flow

| Location | Num Obs | Observed Mean | Computed Mean | Mean Error | RMS Error |
|---|---|---|---|---|---|
| 35 | 1 | 0.03 | 0.01 | 0.024 | 0.024 |
| 53 | 1 | 0.18 | 0.15 | 0.030 | 0.030 |
| 78 | 1 | 0.25 | 0.34 | 0.090 | 0.090 |
| 90 | 1 | 0.21 | 0.25 | 0.040 | 0.040 |
| 102 | 1 | 0.36 | 0.32 | 0.040 | 0.040 |
| 110 | 1 | 0.42 | 0.40 | 0.020 | 0.020 |
| Network | 6 | 0.24 | 0.24 | 0.041 | 0.047 |

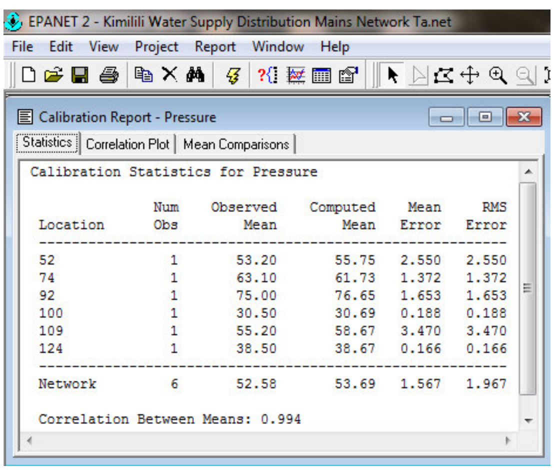
Correlation Between Means: 0.936

**Figure 4. 12: Link Flows Calibration.**

### 4.3.3 Kimilili Water Supply EPANET 2.0 Model Network Validation.

The validation process entailed loading of observed data for a different set of link flows and nodal pressures into the calibrated EPANET 2.0 model network platform as 'flow validation' and 'pressure validation' files respectively, and then running validation of the system.

Successful model validation was achieved when $r$ = 0.994 and 0.993 for nodal point pressures and link flows respectively. According to (Douglas, 2003) for correlation a very strong relationship exists when $r$ = 0.80-1.0. The model was therefore adopted for this research. Figures 4.13 and 4.14 represent the model network validation reports for nodal point pressures and link flows respectively.




Calibration Statistics for Pressure

| Location | Num Obs | Observed Mean | Computed Mean | Mean Error | RMS Error |
|---|---|---|---|---|---|
| 52 | 1 | 53.20 | 55.75 | 2.550 | 2.550 |
| 74 | 1 | 63.10 | 61.73 | 1.372 | 1.372 |
| 92 | 1 | 75.00 | 76.65 | 1.653 | 1.653 |
| 100 | 1 | 30.50 | 30.69 | 0.188 | 0.188 |
| 109 | 1 | 55.20 | 58.67 | 3.470 | 3.470 |
| 124 | 1 | 38.50 | 38.67 | 0.166 | 0.166 |
| Network | 6 | 52.58 | 53.69 | 1.567 | 1.967 |

Correlation Between Means: 0.994

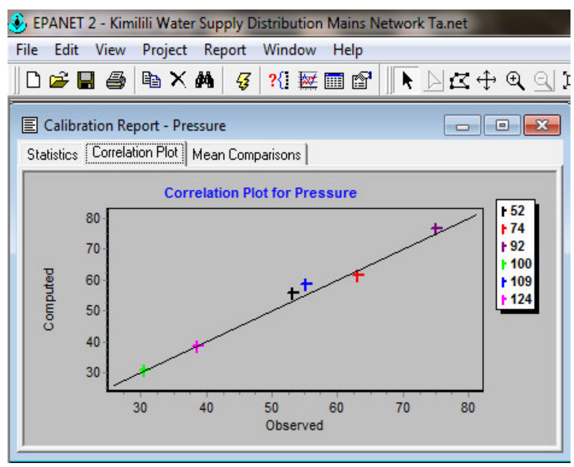
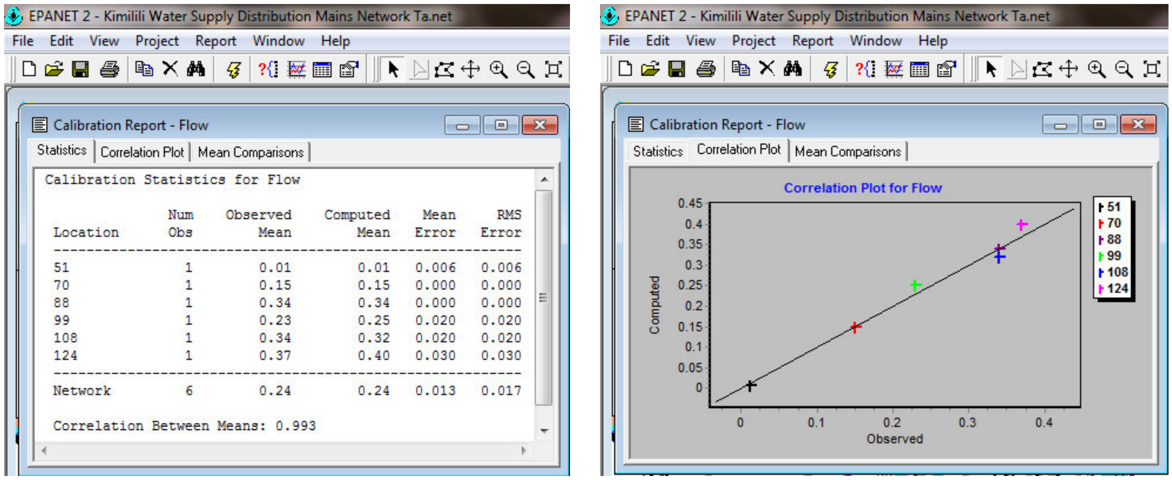



**Figure 4. 13: Nodal Points Validation.**



EPANET 2 - Kimilili Water Supply Distribution Mains Network Ta.net

Calibration Report - Flow

Calibration Statistics for Flow

| Location | Num Obs | Observed Mean | Computed Mean | Mean Error | RMS Error |
|---|---|---|---|---|---|
| 51 | 1 | 0.01 | 0.01 | 0.006 | 0.006 |
| 70 | 1 | 0.15 | 0.15 | 0.000 | 0.000 |
| 88 | 1 | 0.34 | 0.34 | 0.000 | 0.000 |
| 99 | 1 | 0.23 | 0.25 | 0.020 | 0.020 |
| 108 | 1 | 0.34 | 0.32 | 0.020 | 0.020 |
| 124 | 1 | 0.37 | 0.40 | 0.030 | 0.030 |
| Network | 6 | 0.24 | 0.24 | 0.013 | 0.017 |

Correlation Between Means: 0.993



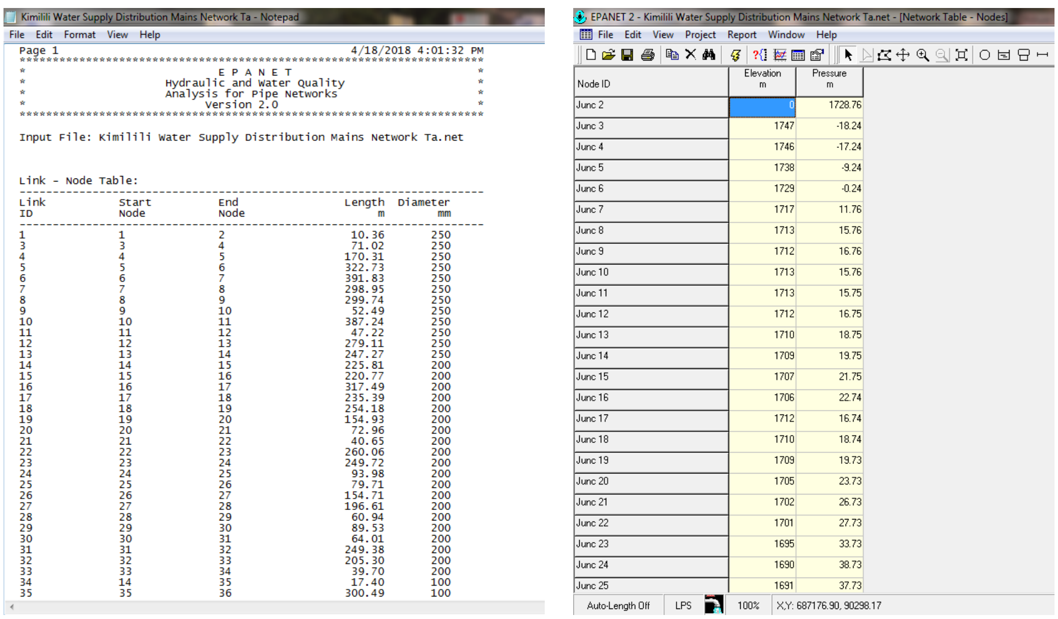
**Figure 4. 14: Link Flows Validation.**

### 4.3.4 Hydraulic simulation of Kimilili water supply distribution network model.

The hydraulic simulation process involved gathering the forecasted monthly water demand into nine demand periods, feeding the periodic demand data into the calibrated and validated EPANET 2.0 distribution network model one at a time, running the simulation and generation of the simulation report in notepad format and saving. Finally the results for the forecasted periodic water demand versus the system water losses for all the nine demand periods were then combined and presented in both tabular and graphical formats.

**(a) Periodic and extended annual zonal nodal demand and pressure simulation.**

While carrying out the periodic hydraulic simulation it was assumed that at any given moment the reservoir water release level was maintained at six meters and the main pipeline network system remained unchanged. Flow rate and pressure head were of major concern as flow rate at each node was expected to be positive in order to meet the water demand requirements otherwise negative flow rate would be an indication of deficiency in meeting water demand required. Positive pressure heads indicated presence of hydrostatic pressure to drive the water to the consumer point while negative pressure head was an indication of deficiency in hydrostatic pressure to further drive water to the consumer points. The flow velocity was not of major concern as the water distribution system was a gravity one thus water losses due to flow velocity were not significant to affect water conveyance to the consumer points. The elevations of the nodes were determined by their global positioning hence fixed. Figure 4.15 shows a sample extract of the hydraulic simulation report.



Kimilili Water Supply Distribution Mains Network Ta - Notepad

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E P A N E T
Hydraulic and Water Quality
Analysis for Pipe Networks
Version 2.0

Input File: Kimilili Water Supply Distribution Mains Network Ta.net

Link - Node Table:

| Link ID | Start Node | End Node | Length m | Diameter mm |
|---|---|---|---|---|
| 1 | 1 | 2 | 10.36 | 250 |
| 3 | 3 | 4 | 71.02 | 250 |
| 4 | 4 | 5 | 170.31 | 250 |
| 5 | 5 | 6 | 322.73 | 250 |
| 6 | 6 | 7 | 391.83 | 250 |
| 7 | 7 | 8 | 298.95 | 250 |
| 8 | 8 | 9 | 299.74 | 250 |
| 9 | 9 | 10 | 52.49 | 250 |
| 10 | 10 | 11 | 387.24 | 250 |
| 11 | 11 | 12 | 47.22 | 250 |
| 12 | 12 | 13 | 279.11 | 250 |
| 13 | 13 | 14 | 247.27 | 250 |
| 14 | 14 | 15 | 225.81 | 200 |
| 15 | 15 | 16 | 220.77 | 200 |
| 16 | 16 | 17 | 317.49 | 200 |
| 17 | 17 | 18 | 235.39 | 200 |
| 18 | 18 | 19 | 254.18 | 200 |
| 19 | 19 | 20 | 154.93 | 200 |
| 20 | 20 | 21 | 72.96 | 200 |
| 21 | 21 | 22 | 40.65 | 200 |
| 22 | 22 | 23 | 260.06 | 200 |
| 23 | 23 | 24 | 249.72 | 200 |
| 24 | 24 | 25 | 93.98 | 200 |
| 25 | 25 | 26 | 79.71 | 200 |
| 26 | 26 | 27 | 154.71 | 200 |
| 27 | 27 | 28 | 196.61 | 200 |
| 28 | 28 | 29 | 60.94 | 200 |
| 29 | 29 | 30 | 89.53 | 200 |
| 30 | 30 | 31 | 64.01 | 200 |
| 31 | 31 | 32 | 249.38 | 200 |
| 32 | 32 | 33 | 205.30 | 200 |
| 33 | 33 | 34 | 39.70 | 200 |
| 34 | 14 | 35 | 17.40 | 100 |
| 35 | 35 | 36 | 300.49 | 100 |

EPANET 2 - Kimilili Water Supply Distribution Mains Network Ta.net - [Network Table - Nodes]

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| Node ID | Elevation m | Pressure m |
|---|---|---|
| Junc 2 | 0 | 1728.76 |
| Junc 3 | 1747 | -18.24 |
| Junc 4 | 1746 | -17.24 |
| Junc 5 | 1738 | -9.24 |
| Junc 6 | 1729 | -0.24 |
| Junc 7 | 1717 | 11.76 |
| Junc 8 | 1713 | 15.76 |
| Junc 9 | 1712 | 16.76 |
| Junc 10 | 1713 | 15.76 |
| Junc 11 | 1713 | 15.75 |
| Junc 12 | 1712 | 16.75 |
| Junc 13 | 1710 | 18.75 |
| Junc 14 | 1709 | 19.75 |
| Junc 15 | 1707 | 21.75 |
| Junc 16 | 1706 | 22.74 |
| Junc 17 | 1712 | 16.74 |
| Junc 18 | 1710 | 18.74 |
| Junc 19 | 1709 | 19.73 |
| Junc 20 | 1705 | 23.73 |
| Junc 21 | 1702 | 26.73 |
| Junc 22 | 1701 | 27.73 |
| Junc 23 | 1695 | 33.73 |
| Junc 24 | 1690 | 38.73 |
| Junc 25 | 1691 | 37.73 |

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**Figure 4. 15: Hydraulic Simulation Report Sample Extract.**

The results for the nine water demand periods hydraulic simulations were as follows;

Table 4.5 shows the hydraulic simulation results for the first water demand period (January 2017 – March 2017).

**Table 4. 5: January 2017 to March 2017 Hydraulic Simulation.**

| Zonal node ID | Demand (l/s) | Head (m) | Pressure (m) | Pressure limit (m) |
|---|---|---|---|---|
| 35 | 2.58 | 1756.36 | 47.36 | 38 m – 51 m |
| 53 | 4.68 | 1755.58 | 48.58 | 38 m – 51 m |
| 81 | 5.49 | 1753.93 | 43.93 | 38 m – 51 m |
| 63 | 4.03 | 1752.55 | 63.55 | 38 m – 51 m |
| 102 | 5.16 | 1751.78 | 54.78 | 38 m – 51 m |
| 110 | 4.52 | 1751.91 | 55.91 | 38 m – 51 m |

The total zonal nodal demand for the period January 2017 to March 2017 was 26.47 l/s and total nodal demand was 28.16 l/s while the entire system demand was 29.37 l/s. System generated demand losses were 1.21 l/s being 4.12 % of the total system demand.

Table 4.6 shows the hydraulic simulation results for the second water demand period (April 2017 - September 2017).

**Table 4. 6: April 2017 to September 2017 Hydraulic Simulation.**

| Zonal node ID | Demand (l/s) | Head (m) | Pressure (m) | Pressure limit (m) |
|---|---|---|---|---|
| 35 | 2.65 | 1756.20 | 47.20 | 38 m – 51 m |
| 53 | 4.79 | 1755.38 | 48.38 | 38 m – 51 m |
| 81 | 5.62 | 1753.66 | 43.66 | 38 m – 51 m |
| 63 | 4.13 | 1752.21 | 63.21 | 38 m – 51 m |
| 102 | 5.29 | 1751.41 | 54.41 | 38 m – 51 m |
| 110 | 4.63 | 1751.55 | 55.55 | 38 m – 51 m |

For the period April 2017 to September 2017, total zonal nodal demand was 27.11 l/s with a total nodal demand of 28.85 l/s while the entire system demand was 30.09 l/s. System generated demand losses were 1.24 l/s being 4.11 % of the total system demand.

Table 4.7 presents hydraulic simulation results for the third water demand period (October 2017 to March 2018).

**Table 4. 7: October 2017 to March 2018 Hydraulic Simulation.**

| Zonal node ID | Demand (l/s) | Head (m) | Pressure (m) | Pressure limit (m) |
|---|---|---|---|---|
| 35 | 2.76 | 1755.91 | 46.91 | 38 m – 51 m |
| 53 | 5.00 | 1555.02 | 48.02 | 38 m – 51 m |
| 81 | 5.87 | 1753.16 | 43.16 | 38 m – 51 m |
| 63 | 4.31 | 1751.60 | 62.60 | 38 m – 51 m |
| 102 | 5.52 | 1750.73 | 53.73 | 38 m – 51 m |
| 110 | 4.83 | 1750.88 | 54.88 | 38 m – 51 m |

There was a total zonal nodal demand of 28.30 l/s with a total nodal demand of 30.09 l/s, while the entire system demand was 31.32 l/s for the period October 2017 to March 2018. The System generated demand losses were 1.23 l/s being 3.93 % of the total system demand.

The hydraulic simulation results for the water demand period April 2018 to September 2018 (fourth) are presented in Table 4.8.

**Table 4. 8: April 2018 to September 2018 Hydraulic Simulation.**

| Zonal node ID | Demand (l/s) | Head (m) | Pressure (m) | Pressure limit (m) |
|---|---|---|---|---|
| 35 | 2.86 | 1755.65 | 46.65 | 38 m – 51 m |
| 53 | 5.18 | 1754.70 | 47.70 | 38 m – 51 m |
| 81 | 6.08 | 1752.71 | 42.71 | 38 m – 51 m |
| 63 | 4.47 | 1751.05 | 62.05 | 38 m – 51 m |
| 102 | 5.72 | 1750.12 | 53.12 | 38 m – 51 m |
| 110 | 5.01 | 1750.28 | 54.28 | 38 m – 51 m |

The generated total zonal nodal demand, total nodal demand and entire system demand for the period April 2018 to Sept 2018 was 29.32 l/s, 31.16 l/s and 32.40 l/s respectively. 1.24 l/s was system generated demand loss being 3.81 % of the entire system demand.

Table 4.9 shows the hydraulic simulation results for the fifth water demand period (October 2018 to March 2019).

**Table 4. 9: October 2018 to March 2019 Hydraulic Simulation.**

| Zonal node ID | Demand (l/s) | Head (m) | Pressure (m) | Pressure limit (m) |
|---|---|---|---|---|
| 35 | 3.01 | 1755.25 | 46.25 | 38 m – 51 m |
| 53 | 5.45 | 1754.22 | 47.22 | 38 m – 51 m |
| 81 | 6.39 | 1752.04 | 42.04 | 38 m – 51 m |
| 63 | 4.70 | 1750.22 | 61.22 | 38 m – 51 m |
| 102 | 6.02 | 1749.20 | 52.20 | 38 m – 51 m |
| 110 | 5.26 | 1749.38 | 53.38 | 38 m – 51 m |

The total system demand for October 2018 to March 2019 period was 33.96 l/s with system demand loss of 1.22 l/s being 3.6 % of the total system demand. The generated total zonal nodal demand and total nodal demand were 30.83 l/s and 32.74 l/s respectively.

The hydraulic simulation results for the sixth water demand period (April 2019 to September 2019) are presented in Table 4.10.

**Table 4. 10: April 2019 to September 2019 Hydraulic Simulation.**

| Zonal node ID | Demand (l/s) | Head (m) | Pressure (m) | Pressure limit (m) |
|---|---|---|---|---|
| 35 | 3.11 | 1754.95 | 45.95 | 38 m – 51 m |
| 53 | 5.64 | 1753.86 | 46.86 | 38 m – 51 m |
| 81 | 6.62 | 1751.54 | 41.54 | 38 m – 51 m |
| 63 | 4.86 | 1749.60 | 60.60 | 38 m – 51 m |
| 102 | 6.23 | 1748.51 | 51.51 | 38 m – 51 m |
| 110 | 5.45 | 1748.70 | 52.70 | 38 m – 51 m |

The total zonal nodal demand for the period April 2019 to Sept 2019 was 31.91 l/s and total nodal demand was 33.87 l/s while the entire system demand was 35.09 l/s. System generated demand losses were 1.22 l/s being 3.48 % of the total system demand.

Table 4.11 presents hydraulic simulation results for the seventh water demand period (October 2019 to March 2019).

**Table 4. 11: October 2019 to March 2020 Hydraulic Simulation.**

| Zonal node ID | Demand (l/s) | Head (m) | Pressure (m) | Pressure limit (m) |
|---|---|---|---|---|
| 35 | 3.22 | 1754.65 | 45.65 | 38 m – 51 m |
| 53 | 5.83 | 1753.49 | 46.49 | 38 m – 51 m |
| 81 | 6.84 | 1751.02 | 41.02 | 38 m – 51 m |
| 63 | 5.03 | 1748.96 | 59.96 | 38 m – 51 m |
| 102 | 6.44 | 1747.81 | 50.81 | 38 m – 51 m |
| 110 | 5.63 | 1748.01 | 52.01 | 38 m – 51 m |

The generated total zonal nodal demand, total nodal demand and entire system demand for the period October 2019 to March 2020 was 32.99 l/s, 35.00 l/s and 36.22 l/s respectively. 1.22 l/s was system generated demand loss being 3.35 % of the entire system demand.

Table 4.12 shows the hydraulic simulation results for the eighth water demand period (April 2020 to September 2020).

**Table 4. 12: April 2020 to September 2020 Hydraulic Simulation.**

| Zonal node ID | Demand (l/s) | Head (m) | Pressure (m) | Pressure limit (m) |
|---|---|---|---|---|
| 35 | 3.22 | 1754.64 | 45.64 | 38 m – 51 m |
| 53 | 5.83 | 1753.48 | 46.48 | 38 m – 51 m |
| 81 | 6.84 | 1751.01 | 41.01 | 38 m – 51 m |
| 63 | 5.03 | 1748.94 | 59.94 | 38 m – 51 m |
| 102 | 6.44 | 1747.78 | 50.79 | 38 m – 51 m |
| 110 | 5.63 | 1747.99 | 51.99 | 38 m – 51 m |

There was a total zonal nodal demand of 33.00 l/s with a total nodal demand of 35.03 l/s, while the entire system demand was 36.25 l/s for the period April 2020 to September 2020. The System generated demand losses were 1.21 l/s being 3.34 % of the total system demand.

The hydraulic simulation results for the ninth water demand period (October 2020 to December 2020) are presented in Table 4.13.

**Table 4. 13: October 2020 to December 2020 Hydraulic Simulation.**

| Zonal node ID | Demand (l/s) | Head (m) | Pressure (m) | Pressure limit (m) |
|---|---|---|---|---|
| 35 | 3.30 | 1754.40 | 45.40 | 38 m – 51 m |
| 53 | 5.98 | 1753.19 | 46.19 | 38 m – 51 m |
| 81 | 7.01 | 1750.60 | 40.60 | 38 m – 51 m |
| 63 | 5.15 | 1748.44 | 64.44 | 38 m – 51 m |
| 102 | 6.59 | 1747.24 | 50.24 | 38 m – 51 m |
| 110 | 5.77 | 1747.44 | 51.44 | 38 m – 51 m |

The total system demand for October 2020 to December 2020 period was 37.11 l/s with system demand loss of 1.21 l/s being 3.27 % of the total system demand. The generated total zonal nodal demand and total nodal demand were 33.80 l/s and 35.90 l/s respectively.

The results for the ten years extended annual demand hydraulic simulations were as presented in Tables 4.14 to Table 4.23.

**Table 4. 14: 21 Hydraulic Simulation**

| Zonal node ID | Demand (l/s) | Head (m) | Pressure (m) | Pressure limit (m) |
|---|---|---|---|---|
| 35 | 3.42 | 1754.20 | 45.20 | 38 m – 51 m |
| 53 | 6.19 | 1754.29 | 47.20 | 38 m – 51 m |
| 81 | 7.26 | 1751.54 | 41.54 | 38 m – 51 m |
| 63 | 5.33 | 1737.00 | 48.00 | 38 m – 51 m |
| 102 | 6.82 | 1744.50 | 47.50 | 38 m – 51 m |
| 110 | 5.97 | 1739.17 | 43.17 | 38 m – 51 m |

The total zonal nodal demand for the year 2021 was 34.99 l/s and total nodal demand was 37.16 l/s while the entire system demand was 38.39 l/s. System generated demand losses were 1.23 l/s being 3.21 % of the total system demand.

**Table 4. 15: 2022 Hydraulic Simulation**

| Zonal node ID | Demand (l/s) | Head (m) | Pressure (m) | Pressure limit (m) |
|---|---|---|---|---|
| 35 | 3.64 | 1754.20 | 45.20 | 38 m – 51 m |
| 53 | 6.60 | 1753.32 | 46.32 | 38 m – 51 m |
| 81 | 7.74 | 1750.21 | 40.21 | 38 m – 51 m |
| 63 | 5.69 | 1737.00 | 48.00 | 38 m – 51 m |
| 102 | 7.28 | 1744.45 | 47.50 | 38 m – 51 m |
| 110 | 6.37 | 1737.42 | 41.42 | 38 m – 51 m |

The total system demand for the year 2022 was 40.86 l/s with system demand loss of 1.22 l/s being 2.99 % of the total system demand. The generated total zonal nodal demand and total nodal demand were 37.32 l/s and 39.64 l/s respectively.

**Table 4. 16: 2023 Hydraulic Simulation**

| Zonal node ID | Demand (l/s) | Head (m) | Pressure (m) | Pressure limit (m) |
|---|---|---|---|---|
| 35 | 3.96 | 1753.42 | 44.42 | 38 m – 51 m |
| 53 | 7.17 | 1751.74 | 44.74 | 38 m – 51 m |
| 81 | 8.40 | 1748.12 | 39.12 | 38 m – 51 m |
| 63 | 6.17 | 1737.00 | 48.00 | 38 m – 51 m |
| 102 | 7.90 | 1743.41 | 46.41 | 38 m – 51 m |
| 110 | 6.92 | 1734.70 | 39.70 | 38 m – 51 m |

The total system demand for the year 2023 was 44.25 l/s with system demand loss of 1.21 l/s being 2.73% of the total system demand. The generated total zonal nodal demand and total nodal demand were 40.52 l/s and 43.04 l/s respectively.

**Table 4. 17: 2024 Hydraulic Simulation**

| Zonal node ID | Demand (l/s) | Head (m) | Pressure (m) | Pressure limit (m) |
|---|---|---|---|---|
| 35 | 3.96 | 1753.93 | 44.93 | 38 m – 51 m |
| 53 | 7.17 | 1752.12 | 45.12 | 38 m – 51 m |
| 81 | 8.40 | 1748.20 | 39.20 | 38 m – 51 m |
| 63 | 6.17 | 1737.00 | 48.00 | 38 m – 51 m |
| 102 | 7.90 | 1743.10 | 46.10 | 38 m – 51 m |
| 110 | 6.92 | 1734.41 | 39.41 | 38 m – 51 m |

The total system demand for the year 2024 was 46.16 l/s with system demand loss of 1.21 l/s being 2.62 % of the total system demand. The generated total zonal nodal demand and total nodal demand were 42.32 l/s and 44.95 l/s respectively.

**Table 4. 18: 2025 Hydraulic Simulation**

| Zonal node ID | Demand (l/s) | Head (m) | Pressure (m) | Pressure limit (m) |
|---|---|---|---|---|
| 35 | 4.30 | 1753.87 | 44.87 | 38 m – 51 m |
| 53 | 7.79 | 1751.92 | 44.92 | 38 m – 51 m |
| 81 | 9.13 | 1747.71 | 38.71 | 38 m – 51 m |
| 63 | 6.71 | 1737.00 | 48.00 | 38 m – 51 m |
| 102 | 8.58 | 1742.22 | 45.22 | 38 m – 51 m |
| 110 | 7.51 | 1733.56 | 38.56 | 38 m – 51 m |

The total zonal nodal demand for the year 2025 was 44.02 l/s and total nodal demand was 46.75 l/s while the entire system demand was 47.96 l/s. System generated demand losses were 1.21 l/s being 2.52 % of the total system demand.

**Table 4. 19: 2026 Hydraulic Simulation**

| Zonal node ID | Demand (l/s) | Head (m) | Pressure (m) | Pressure limit (m) |
|---|---|---|---|---|
| 35 | 4.58 | 1753.12 | 44.12 | 38 m – 51 m |
| 53 | 8.29 | 1750.94 | 43.94 | 38 m – 51 m |
| 81 | 9.72 | 1746.20 | 38.20 | 38 m – 51 m |
| 63 | 7.14 | 1737.00 | 48.00 | 38 m – 51 m |
| 102 | 9.14 | 1740.03 | 43.03 | 38 m – 51 m |
| 110 | 8.01 | 1731.41 | 38.41 | 38 m – 51 m |

The total zonal nodal demand for the year 2026 was 46.87 l/s and total nodal demand was 49.78 l/s while the entire system demand was 50.99 l/s. System generated demand losses were 1.21 l/s being 2.37 % of the total system demand.

**Table 4. 20: 2027 Hydraulic Simulation**

| Zonal node ID | Demand (l/s) | Head (m) | Pressure (m) | Pressure limit (m) |
|---|---|---|---|---|
| 35 | 5.00 | 1750.54 | 41.54 | 38 m – 51 m |
| 53 | 9.06 | 1747.99 | 40.99 | 38 m – 51 m |
| 81 | 10.62 | 1742.41 | 34.41 | 38 m – 51 m |
| 63 | 7.80 | 1737.00 | 48.00 | 38 m – 51 m |
| 102 | 9.99 | 1735.14 | 38.14 | 38 m – 51 m |
| 110 | 8.74 | 1726.59 | 33.59 | 38 m – 51 m |

The total system demand for the year 2027 was 55.58 l/s with system demand loss of 1.18 l/s being 2.12 % of the total system demand. The generated total zonal nodal demand and total nodal demand were 51.22 l/s and 54.40 l/s respectively.

**Table 4. 21: 2028 Hydraulic Simulation**

| Zonal node ID | Demand (l/s) | Head (m) | Pressure (m) | Pressure limit (m) |
|---|---|---|---|---|
| 35 | 5.35 | 1748.25 | 39.25 | 38 m – 51 m |
| 53 | 9.69 | 1745.38 | 38.38 | 38 m – 51 m |
| 81 | 11.35 | 1739.06 | 31.06 | 38 m – 51 m |
| 63 | 8.34 | 1733.78 | 44.78 | 38 m – 51 m |
| 102 | 10.67 | 1730.83 | 33.83 | 38 m – 51 m |
| 110 | 9.35 | 1722.34 | 29.34 | 38 m – 51 m |

The total system demand for the year 2028 was 59.30 l/s with system demand loss of 1.15 l/s being 1.94 % of the total system demand. The generated total zonal nodal demand and total nodal demand were 54.75 l/s and 58.15 l/s respectively.

**Table 4. 22: 2029 Hydraulic Simulation**

| Zonal node ID | Demand (l/s) | Head (m) | Pressure (m) | Pressure limit (m) |
|---|---|---|---|---|
| 35 | 5.58 | 1746.60 | 37.60 | 38 m – 51 m |
| 53 | 10.11 | 1743.50 | 36.50 | 38 m – 51 m |
| 81 | 11.85 | 1736.66 | 28.66 | 38 m – 51 m |
| 63 | 8.70 | 1730.95 | 41.95 | 38 m – 51 m |
| 102 | 11.14 | 1727.76 | 30.76 | 38 m – 51 m |
| 110 | 9.75 | 1719.31 | 26.31 | 38 m – 51 m |

The total zonal nodal demand for the year 2029 was 57.13 l/s and total nodal demand was 60.68 l/s while the entire system demand was 61.82 l/s. System generated demand losses were 1.14 l/s being 1.84 % of the total system demand.

**Table 4. 23: 2030 Hydraulic Simulation**

| Zonal node ID | Demand (l/s) | Head (m) | Pressure (m) | Pressure limit (m) |
|---|---|---|---|---|
| 35 | 5.73 | 1745.49 | 36.49 | 38 m – 51 m |
| 53 | 10.38 | 1742.23 | 35.23 | 38 m – 51 m |
| 81 | 12.17 | 1735.05 | 27.05 | 38 m – 51 m |
| 63 | 8.94 | 1729.04 | 40.04 | 38 m – 51 m |
| 102 | 11.44 | 1725.70 | 28.70 | 38 m – 51 m |
| 110 | 10.02 | 1717.28 | 24.28 | 38 m – 51 m |

The total system demand for the year 2030 was 63.44 l/s with system demand loss of 1.12 l/s being 1.77 % of the total system demand. The generated total zonal nodal demand and total nodal demand were 58.67 l/s and 62.32 l/s respectively.

**(b) Sensitivity Analysis Results**

In order to recognize the possible policy and theoretical implications of the results, it was considered important to conduct sensitivity analysis. Sensitivity of the model in estimating system water losses was investigated through estimating water demand against water losses over the years. Two scenarios were examined and the results were;

**(i) Periodic System Water Demand versus System Water Losses**

The generated periodic hydraulic simulation results in Table 4.24 and Figure 4.16 indicate that a general relationship between system water demand and system water losses can be demonstrated. From the results it is generally established that system water losses reduces with increase in system water demand. When zonal nodal water demand increases the pressure for the respective zonal node reduces, consequently the total zonal node pressure reduction leads to significant system pressure reduction, hence reduction in the amount of water lost due to system pressures.

**Table 4. 24: Periodic System Water Demand Versus System Water Losses.**

| Period | System Water Demand (l/s) | System Losses (l/s) | % Losses |
|---|---|---|---|
| Jan 2017 - March 2017 | 29.4 | 1.24 | 4.22 |
| April 2017 - Sep 2017 | 30.09 | 1.24 | 4.11 |
| Oct 2017 - March 2018 | 31.32 | 1.23 | 3.93 |
| April 2018 - Sep 2018 | 32.4 | 1.24 | 3.81 |
| Oct 2018 - March 2019 | 33.96 | 1.22 | 3.60 |
| April 2019 - Sep 2019 | 35.09 | 1.22 | 3.48 |
| Oct 2019 - March 2020 | 36.22 | 1.22 | 3.35 |
| April 2020 - Sep 2020 | 36.25 | 1.21 | 3.34 |
| Oct 2020 - Dec 2020 | 37.11 | 1.21 | 3.27 |

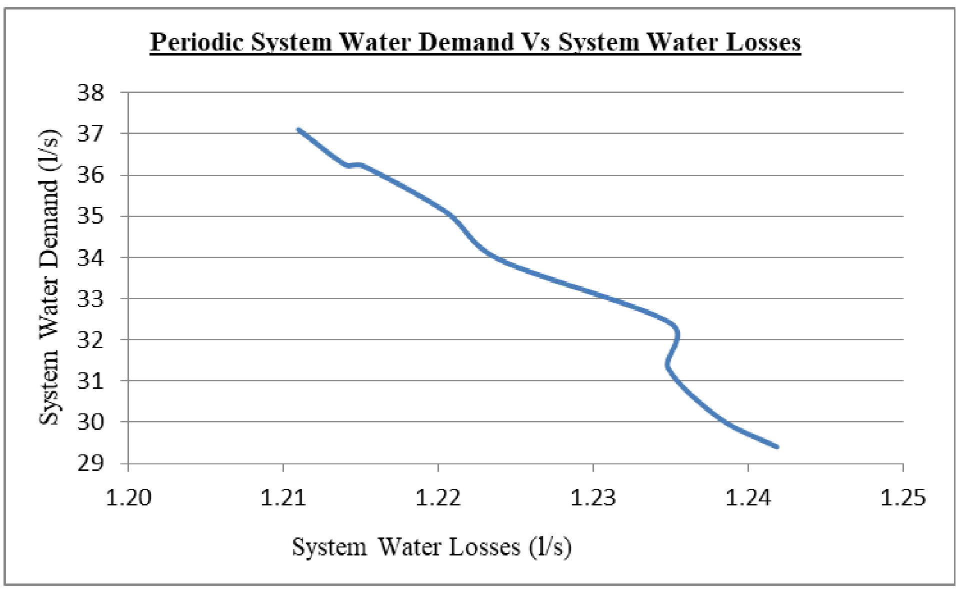


**Figure 4. 16: Periodic System Water Demand Versus System Water Losses.**

Detailed analysis of the results, established the relationship between system water demand and system water losses mathematically. Through generation of the best curve of fit for the graph of periodic system water demand against system water losses using excel, it is established that the general relationship between periodic system water demand and system water losses is an exponential function defined by Equation 4.3 and Figure 4.17.

$$y = 256394e^{-7.296x} \quad \text{……………………………………………Equation (4.3)}$$

Where;

y = periodic system water demand (l/s)

e = exponent (2.718281828459)

x = periodic system water losses (l/s)

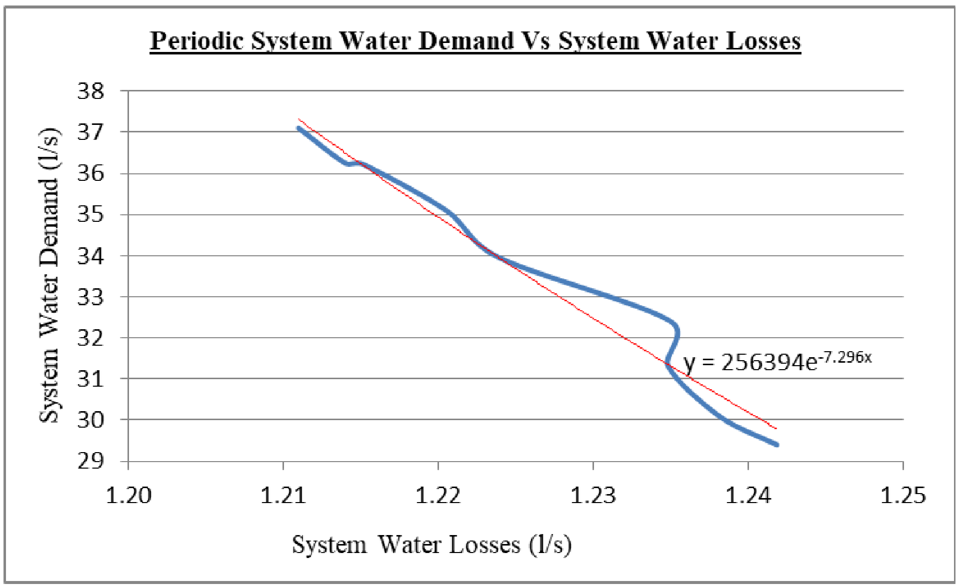


**Figure 4. 17: Mathematical Relationship Between Periodic System Water Demand Versus System Water Losses.**

**(ii) Periodic System Water Demand versus Percentage System Water Losses**

An analysis of the results for periodic system water demand and percentage water losses was carried out by plotting a graph of periodic system water demand against periodic percentage system water losses as presented in Figure 4.18.

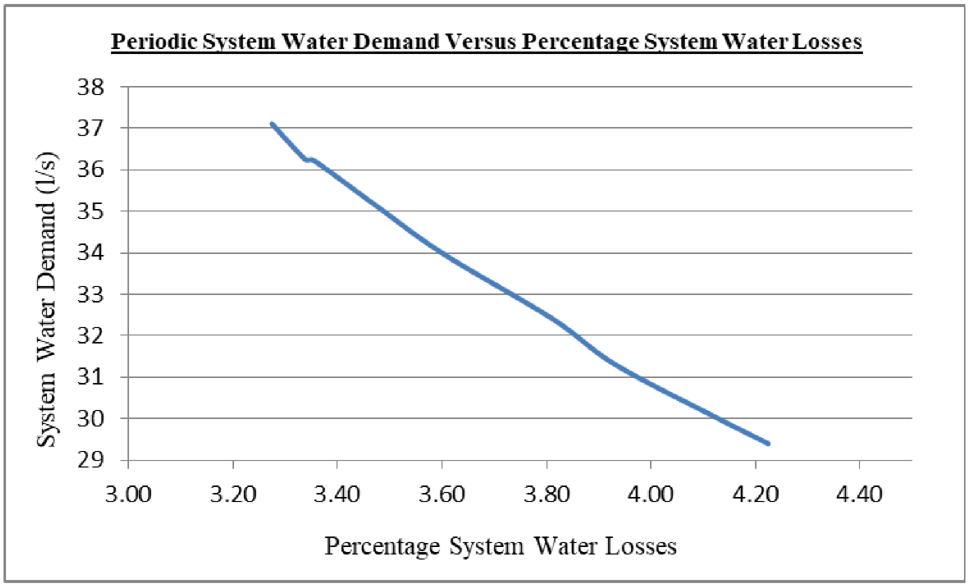


**Figure 4. 18: Periodic System Water Demand Versus Percentage System Water Losses.**

Further analysis of the graph of periodic system water demand against periodic percentage system water losses by drawing the best curve of fit using excel established that the general relationship between periodic system water demand and periodic percentage system water losses is a polynomial function of order two defined by Equation 4.4 and Figure 4.19.

$y=1.8503x^2 - 21.882x + 88.808$ ……………….……….………...…………Equation (4.4)

Where;

y = periodic system water demand (l/s)

x = periodic percentage system water losses (%)

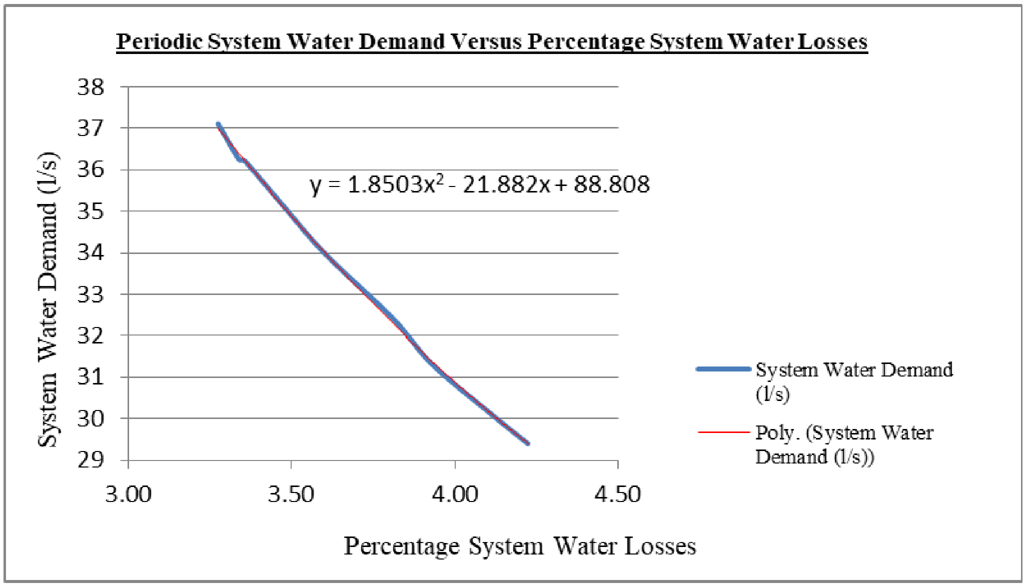


**Figure 4. 19: Mathematical Relationship Between Periodic System Water Demand And Percentage System Water Losses.**

### 4.4 Establishment of optimal reservoir water release levels

The process of establishing the optimal reservoir water release levels for Kimilili water supply scheme pipeline distribution network entailed determining the required nodal points pressure ranges and then determining the reservoir water release levels that would result to the generation of the required nodal point pressures at any given system water demand.

#### 4.4.1 Determination of required nodal point's pressures

According to (NZOWASCO, 2015) annual report, the majority (72%) of Kimilili water supply distribution network is made up of unplasticized polyvinyl chloride (uPVC) class 'B' pipes, 16% of the pipeline is made up of uPVC class 'C' pipes, 10.2% of the pipeline is made up of galvanized iron (GI) class 'B' pipes and the remaining 1.8% of the pipeline is made up of asbestos cement (AC). According to the international organization for standardization (ISO 1452-2:2009) standards, maximum working pressure head for uPVC class 'B' pipes is 60 meter water head (6 bars) and the average working pressure is 45 meter water head (4.5) bars. Considering 15% working allowance, the optimal pressure range is 4.5±15% (3.8 bars – 5.1 bars). Therefore, the adopted required zonal nodal point's pressure range for this study was 3.8 bars to 5.1 bars.

### 4.4.2 Determination of the optimal reservoir water release levels

The process of determining the optimal reservoir water release levels of Kimilili water supply scheme pipeline distribution network for the various water demand periods involved grouping the MATLAB R2014a (ANN) model forecasted 48 months water demand data into nine water demand period, calculating the average periodic water demand. Finally running iterative hydraulic simulations of Kimilili water supply scheme for each of the nine water demand periods with varying reservoir water release levels in combination of various valve types and valve fixed status respectively and determining the combination that generated zonal nodal pressures between 3.8 bars to 5.1 bars for all the six zonal nodes to be the optimal reservoir water release level.

### 4.4.3 Periodic and Extended Annual Optimal Reservoir Water Release levels

The hydraulic simulation results generated for the established optimal reservoir water release levels for each of the nine water demand periods and ten year extended annual demand are as presented in Tables (4.25 to 4.43).

Table 4.25 shows the hydraulic simulation optimal reservoir water release level results for the first water demand period (January 2017 to March 2017).

**Table 4. 25: January 2017 to March 2017 Optimal Reservoir Water Release Level.**

| Zonal Node ID | Pressure (m) | Valve ID | Type | Status | Setting (m, l/s) |
|---|---|---|---|---|---|
| RWRL (m) | 7.7 | 2 | FCV | Open | 29.37 |
| 35 | 44.00 | 34 | PRV | Active | 44.00 |
| 53 | 45.58 | 52 | PRV | Active | 45.58 |
| 81 | 45.63 | 77 | PRV | Active | 46.00 |
| 63 | 48.00 | 89 | PRV | Active | 48.00 |
| 102 | 47.50 | 101 | PRV | Active | 47.50 |
| 110 | 47.62 | 109 | PBV | Active | 10.00 |

The optimal reservoir water release level for January 2017 to March 2017 water demand period is seven point seven (7.7) meters with the main flow control valve no. 2 being set at 29.37l/s. Pressure release valves (34, 52, 77, 89, 101) serving zonal node nos. 35, 53, 81, 63 and 102 have to be active and set at 4.4 bars, 4.5bars, 4.6 bars, 4.8 bars and 4.75 bars respectively while PBV no. 109 serving zonal node no.110 need to be set at 10 meters.

Table 4.26 presents hydraulic simulation optimal reservoir water release level results for the second water demand period (April 2017 to September 2017).

**Table 4. 26: April 2017 to September 2017 Optimal Reservoir Water Release Level.**

| Zonal Node ID | Pressure (m) | Valve ID | Type | Status | Setting (m) |
|---|---|---|---|---|---|
| RWRL (m) | 8.0 | 2 | FCV | Active | 30.09 |
| 35 | 45.20 | 34 | PRV | Active | 45.20 |
| 53 | 48.82 | 52 | PRV | Active | 50.00 |
| 81 | 44.10 | 77 | PRV | Active | 46.00 |
| 63 | 48.00 | 89 | PRV | Active | 48.00 |
| 102 | 47.50 | 101 | PRV | Active | 47.50 |
| 110 | 45.99 | 109 | PBV | Active | 10.00 |

The optimal reservoir water release level for April 2017 to September 2017 water demand period is eight (8.0) meters with the main flow control valve no. 2 being set at 30.09l/s. Pressure release valves (34, 52, 77, 89, 101) serving zonal node nos. 35, 53, 81, 63 and 102 have to be active and set at 4.5 bars, 5.0 bars, 4.6 bars, 4.8 bars and 4.75 bars respectively while PBV no. 109 serving zonal node no.110 need to be set at 10 meters.

The hydraulic simulation optimal reservoir water release level results for the third water demand period (October 2017 to March 2018) are presented in Table 4.27.

**Table 4. 27: October 2017 to March 2018 Optimal Reservoir Water Release Level.**

| Zonal Node ID | Pressure (m) | Valve ID | Type | Status | Setting (m) |
|---|---|---|---|---|---|
| RWRL (m) | 8.0 | 2 | FCV | Active | 31.32 |
| 35 | 44.28 | 34 | PRV | Active | 45.20 |
| 53 | 47.65 | 52 | PRV | Active | 50.00 |
| 81 | 42.79 | 77 | PRV | Active | 46.00 |
| 63 | 48.00 | 89 | PRV | Active | 48.00 |
| 102 | 47.50 | 101 | PRV | Active | 47.50 |
| 110 | 44.51 | 109 | PBV | Active | 10.00 |

The optimal reservoir water release level for October 2017 to March 2018 water demand period is eight (8.0) meters with all the control valves setting for April 2017 to September 2017 being maintained except FCV no.2 being adjusted to 31.32l/s. The zonal nodes with the highest and the lowest pressure are node numbers 63 (4.8bars) 81 (4.27bars) respectively.

Table 4.28 shows the hydraulic simulation optimal reservoir water release level results for the fourth water demand period (April 2018 to September 2018).

**Table 4. 28: April 2018 to September 2018 Optimal Reservoir Water Release Level.**

| Zonal Node ID | Pressure (m) | Valve ID | Type | Status | Setting (m) |
|---|---|---|---|---|---|
| RWRL (m) | 8.0 | 2 | FCV | Active | 32.40 |
| 35 | 45.20 | 34 | PRV | Active | 45.20 |
| 53 | 49.23 | 52 | PRV | Active | 50.00 |
| 81 | 43.24 | 77 | PRV | Active | 46.00 |
| 63 | 48.00 | 89 | PRV | Active | 48.00 |
| 102 | 47.50 | 101 | PRV | Active | 47.50 |
| 110 | 44.81 | 109 | PBV | Active | 10.00 |

The optimal reservoir water release level for April 2018 to September 2018 water demand period is eight (8.0) meters with all the control valves setting for the immediate previous period (October 2017 to March 2018) being maintained except FCV no.2 being adjusted to 32.401/s to accommodate the system water demand. The zonal nodes with the highest and the lowest pressure are node numbers 53 (4.92bars) 81 (4.32bars) respectively. Zonal node pressures have increased for April 2018 to September 2018 period as compared to the immediate previous period, this is attributed to the increase in the system water demand and as a result increase in the flow velocities (from 0.64m/s to 0.66m/s) since the pipeline cross sectional areas are fixed.

Table 4.29 presents hydraulic simulation optimal reservoir water release level results for the fifth water demand period (October 2018 to March 2019).

**Table 4. 29: October 2018 to March 2019 Optimal Reservoir Water Release Level.**

| Zonal Node ID | Pressure (m) | Valve ID | Type | Status | Setting (m) |
|---|---|---|---|---|---|
| RWRL (m) | 8.2 | 2 | FCV | Active | 33.96 |
| 35 | 45.20 | 34 | PRV | Active | 45.20 |
| 53 | 46.77 | 52 | PRV | Active | 50.00 |
| 81 | 41.59 | 77 | PRV | Active | 46.00 |
| 63 | 48.00 | 89 | PRV | Active | 48.00 |
| 102 | 47.50 | 101 | PRV | Active | 47.50 |
| 110 | 43.93 | 109 | PBV | Active | 9.00 |

The optimal reservoir water release level for October 2018 to March 2019 water demand period is eight point two (8.2) meters with all the control valves setting for the immediate previous period (April 2018 to September 2018) being maintained except for FCV no.2 being adjusted to 33.96l/s to accommodate the system water demand and PBV 109 that need to be adjusted to 9m. The zonal nodes with the highest and the lowest pressure are node numbers 63 (4.8bars) and 81 (4.15bars) respectively. Zonal node pressures have slightly decreased for October 2018 to March 2019 period as compared to the immediate previous period, thus the need of reducing the setting of PBV from 10m to 9m. There is an increase in the flow velocities from 0.66m/s to 0.69m/s.

Table 4.30 presents hydraulic simulation optimal reservoir water release level results for the sixth water demand period (April 2019 to September 2019).

**Table 4. 30: April 2019 to September 2019 Optimal Reservoir Water Release Level.**

| Zonal Node ID | Pressure (m) | Valve ID | Type | Status | Setting (m) |
|---|---|---|---|---|---|
| RWRL (m) | 8.2 | 2 | FCV | Active | 35.09 |
| 35 | 45.20 | 34 | PRV | Active | 45.20 |
| 53 | 46.33 | 52 | PRV | Active | 50.00 |
| 81 | 41.01 | 77 | PRV | Active | 46.00 |
| 63 | 48.00 | 89 | PRV | Active | 48.00 |
| 102 | 47.50 | 101 | PRV | Active | 47.50 |
| 110 | 43.17 | 109 | PBV | Active | 9.00 |

For the period April 2019 to September 2019, the optimal reservoir water release level is eight point two (8.2) meters with all the control valves setting for October 2018 to March 2019 being maintained except FCV no.2 being adjusted to 35.09l/s to accommodate the system water demand. The zonal nodes with the highest and the lowest pressure are node numbers 63 (4.8bars) 81 (4.1bars) respectively. The flow velocities increased from 0.69m/s to 0.71m/s.

Table 4.31 shows the hydraulic simulation optimal reservoir water release level results for the seventh water demand period (October 2019 to March 2020).

**Table 4. 31: October 2019 to March 2020 Optimal Reservoir Water Release Level.**

| Zonal Node ID | Pressure (m) | Valve ID | Type | Status | Setting (m) |
|---|---|---|---|---|---|
| RWRL (m) | 8.5 | 2 | FCV | Active | 36.23 |
| 35 | 45.20 | 34 | PRV | Active | 45.20 |
| 53 | 46.39 | 52 | PRV | Active | 50.00 |
| 81 | 40.92 | 77 | PRV | Active | 46.00 |
| 63 | 48.00 | 89 | PRV | Active | 48.00 |
| 102 | 47.50 | 101 | PRV | Active | 47.50 |
| 110 | 42.90 | 109 | PBV | Active | 9.00 |

The optimal reservoir water release level for October 2019 to March 2020 water demand period is eight point five (8.5) meters with all the control valves setting for the immediate previous period (April 2019 to September 2019) being maintained except FCV no.2 being adjusted to 36.221/s to accommodate the system water demand. The zonal nodes with the highest and the lowest pressure are node numbers 63 (4.8bars) and 81 (4.09bars) respectively. There is slight increase in Zonal node pressures as compared to the immediate previous period (April 2019 to September 2019), this is attributed to the increase in the system water demand and as a result increase in the flow velocities (from 0.71m/s to 0.74m/s) since the pipeline cross sectional areas are fixed.

The hydraulic simulation optimal reservoir water release level results for the eighth water demand period (April 2020 to September 2020) are presented in Table 4.32.

**Table 4. 32: April 2020 to September 2020 Optimal Reservoir Water Release Level.**

| Zonal Node ID | Pressure (m) | Valve ID | Type | Status | Setting (m) |
|---|---|---|---|---|---|
| RWRL (m) | 8.5 | 2 | FCV | Active | 36.27 |
| 35 | 45.20 | 34 | PRV | Active | 45.20 |
| 53 | 46.35 | 52 | PRV | Active | 50.00 |
| 81 | 40.88 | 77 | PRV | Active | 46.00 |
| 63 | 48.00 | 89 | PRV | Active | 48.00 |
| 102 | 47.50 | 101 | PRV | Active | 47.50 |
| 110 | 42.86 | 109 | PBV | Active | 9.00 |

The optimal reservoir water release level for April 2020 to September 2020 water demand period is eight point five (8.5) meters with all the control valves setting for the immediate previous period (October 2019 to March 2020) being maintained except for FCV no.2 being adjusted to 36.25l/s to accommodate the system water demand. The zonal nodes with the highest and the lowest pressure are node numbers 63 (4.8bars) 81 (4.08bars) respectively. There is a slight increase in Zonal node pressures for April 2020 to September 2020 period as compared to the immediate previous period, but the overall flow is maintained at 0.74m/s.

Table 4.33 presents hydraulic simulation optimal reservoir water release level results for the ninth water demand period (October 2020 to December 2020).

**Table 4. 33: October 2020 to December 2020 Optimal Reservoir Water Release Level.**

| Zonal Node ID | Pressure (m) | Valve ID | Type | Status | Setting (m) |
|---|---|---|---|---|---|
| RWRL (m) | 8.7 | 2 | FCV | Active | 37.11 |
| 35 | 45.00 | 34 | PRV | Active | 45.20 |
| 53 | 45.79 | 52 | PRV | Active | 50.00 |
| 81 | 40.20 | 77 | PRV | Active | 46.00 |
| 63 | 48.00 | 89 | PRV | Active | 48.00 |
| 102 | 47.50 | 101 | PRV | Active | 47.50 |
| 110 | 42.05 | 109 | PBV | Active | 9.00 |

The optimal reservoir water release level for October 2020 to December 2020 water demand period is eight point seven (8.7) meters with all the control valves setting for the immediate previous period (April 2020 to September 2020) being maintained except FCV no.2 being adjusted to 37.111/s to accommodate the system water demand. The zonal nodes with the highest and the lowest pressure are node numbers 63 (4.8bars) 81

(4.02 bars) respectively. Compared to the immediate previous period, the zonal node pressures decreased while the overall flow velocity increased to 0.76m/s.

Table 4.34 presents hydraulic simulation optimal reservoir water release level results for the extended demand year 2021.

**Table 4. 34: Extended Year 2021 Optimal Reservoir Water Release Level.**

| Zonal Node ID | Pressure (m) | Valve ID | Type | Status | Setting (m) |
|---|---|---|---|---|---|
| RWRL (m) | 8.70 | 2 | TCV | Active | 38.39 |
| 35 | 45.20 | 34 | PRV | Active | 45.20 |
| 53 | 47.20 | 52 | PRV | Active | 50.00 |
| 81 | 41.54 | 77 | PRV | Active | 46.00 |
| 63 | 48.00 | 89 | PRV | Active | 48.00 |
| 102 | 47.50 | 101 | PRV | Active | 47.50 |
| 110 | 43.17 | 109 | PBV | Active | 9.00 |

The optimal reservoir water release level for extended year 2021 water demand period is eight point seven (8.7) meters with the main throttle control valve no. 2 being set at 38.39 l/s. Pressure release valves (34, 52, 77, 89, 101) serving zonal node nos. 35, 53, 81, 63 and 102 have to be active and set at 4.52 bars, 5.0 bars, 4.6 bars, 4.8 bars and 4.75 bars respectively while PBV no. 109 serving zonal node no.110 need to be set at 9 meters. The overall flow velocity is 0.78m/s with all the six zonal nodes being subjected to pressures between 4.15 bars to 4.8 bars which are within the required pressures range.

Table 4.35 presents hydraulic simulation optimal reservoir water release level results for the extended demand year 2022.

**Table 4. 35: Extended Year 2022 Optimal Reservoir Water Release Level.**

| Zonal Node ID | Pressure (m) | Valve ID | Type | Status | Setting (m) |
|---|---|---|---|---|---|
| RWRL (m) | 8.90 | 2 | TCV | Active | 40.86 |
| 35 | 45.20 | 34 | PRV | Active | 45.20 |
| 53 | 46.32 | 52 | PRV | Active | 50.00 |
| 81 | 40.21 | 77 | PRV | Active | 46.00 |
| 63 | 48.00 | 89 | PRV | Active | 48.00 |
| 102 | 47.50 | 101 | PRV | Active | 47.50 |
| 110 | 41.42 | 109 | PBV | Active | 9.00 |

The optimal reservoir water release level for extended year 2022 water demand period is eight point nine (8.9) meters with all the control valves setting for the immediate previous period (2021) being maintained except TCV no.2 being adjusted to 40.86 1/s to accommodate the system water demand. The zonal nodes with the highest and the lowest pressure are node numbers 63 (4.8bars) and 81 (4.02 bars) respectively. Compared to the immediate previous period, the zonal node pressures decreased while the overall flow velocity increased to 0.83m/s.

Table 4.36 presents hydraulic simulation optimal reservoir water release level results for the extended demand year 2023.

**Table 4. 36: Extended Year 2023 Optimal Reservoir Water Release Level.**

| Zonal Node ID | Pressure (m) | Valve ID | Type | Status | Setting (m) |
|---|---|---|---|---|---|
| RWRL (m) | 9.00 | 2 | TCV | Active | 44.25 |
| 35 | 44.42 | 34 | PRV | Active | 45.20 |
| 53 | 44.74 | 52 | PRV | Active | 50.00 |
| 81 | 39.12 | 77 | PRV | Active | 46.00 |
| 63 | 48.00 | 89 | PRV | Active | 48.00 |
| 102 | 46.41 | 101 | PRV | Active | 47.50 |
| 110 | 39.70 | 109 | PBV | Active | 9.00 |

The optimal reservoir water release level for extended year 2023 water demand period is nine point zero (9.0) meters with all the control valves setting for the immediate previous period (2022) being maintained except for TCV no.2 being adjusted to 44.25 l/s to accommodate the system water demand. The zonal nodes with the highest and the lowest pressure are node numbers 63 (4.8bars) 81 (3.91bars) respectively. There is a slight decrease in Zonal node pressures for 2023 period as compared to the immediate previous period, but the overall flow is increased to 0.90m/s.

Table 4.37 presents hydraulic simulation optimal reservoir water release level results for the extended demand year 2024.

**Table 4. 37: Extended Year 2024 Optimal Reservoir Water Release Level.**

| Zonal Node ID | Pressure (m) | Valve ID | Type | Status | Setting (m) |
|---|---|---|---|---|---|
| RWRL (m) | 10.40 | 2 | TCV | Active | 46.16 |
| 35 | 44.93 | 34 | PRV | Active | 45.20 |
| 53 | 45.12 | 52 | PRV | Active | 50.00 |
| 81 | 39.20 | 77 | PRV | Active | 46.00 |
| 63 | 48.00 | 89 | PRV | Active | 48.00 |
| 102 | 46.10 | 101 | PRV | Active | 47.50 |
| 110 | 39.41 | 109 | PBV | Active | 9.00 |

The optimal reservoir water release level for extended year 2024 water demand period is ten point four (10.4) meters with the main TCV no. 2 being set at 46.16 l/s. Pressure release valves (34, 52, 77, 89, 101) serving zonal node nos. 35, 53, 81, 63 and 102 have to be active and set at 4.52 bars, 5.0 bars, 4.6 bars, 4.8 bars and 4.75 bars respectively while PBV no. 109 serving zonal node no.110 need to be set at 9 meters. The overall flow velocity is 0.94m/s with all the six zonal nodes being subjected to pressures between 3.92 bars to 4.8 bars which are within the required pressure range.

Table 4.38 presents hydraulic simulation optimal reservoir water release level results for the extended demand year 2025.

**Table 4. 38: Extended Year 2025 Optimal Reservoir Water Release Level.**

| Zonal Node ID | Pressure (m) | Valve ID | Type | Status | Setting (m) |
|---|---|---|---|---|---|
| RWRL (m) | 11.20 | 2 | TCV | Active | 47.96 |
| 35 | 44.87 | 34 | PRV | Active | 45.20 |
| 53 | 44.92 | 52 | PRV | Active | 50.00 |
| 81 | 38.71 | 77 | PRV | Active | 46.00 |
| 63 | 48.00 | 89 | PRV | Active | 48.00 |
| 102 | 45.22 | 101 | PRV | Active | 47.50 |
| 110 | 38.56 | 109 | PBV | Active | 9.00 |

For the extended year 2025 water demand period, the optimal reservoir water release level is eleven point two (11.2) meters with all the control valves setting for year 2024 being maintained except TCV no.2 being adjusted to 47.96 l/s to accommodate the system water demand. The zonal nodes with the highest and the lowest pressure are node numbers 63 (4.8bars) and 110 (3.85bars) respectively. All the six zonal nodes are subjected to pressures that are within the required range of 3.8 bars to 5.1 bars with overall flow velocities of 0.98m/s.

Table 4.39 presents hydraulic simulation optimal reservoir water release level results for the extended demand year 2026.

**Table 4. 39: Extended Year 2026 Optimal Reservoir Water Release Level.**

| Zonal Node ID | Pressure (m) | Valve ID | Type | Status | Setting (m) |
|---|---|---|---|---|---|
| RWRL (m) | 12.00 | 2 | TCV | Active | 50.99 |
| 35 | 44.12 | 34 | PRV | Active | 45.20 |
| 53 | 43.94 | 52 | PRV | Active | 50.00 |
| 81 | 38.20 | 77 | PRV | Active | 46.00 |
| 63 | 48.00 | 89 | PRV | Active | 48.00 |
| 102 | 43.03 | 101 | PRV | Active | 47.50 |
| 110 | 38.41 | 109 | PBV | Active | 9.00 |

The optimal reservoir water release level for extended year 2026 water demand period is twelve point zero (12.0) meters with the main TCV no. 2 being set at 50.99 l/s. Pressure release valves (34, 52, 77, 89, 101) serving zonal node nos. 35, 53, 81, 63 and 102 have to be active and set at 4.52 bars, 5.0 bars, 4.6 bars, 4.8 bars and 4.75 bars respectively while PBV no. 109 serving zonal node no.110 need to be set at 9 meters. The overall flow velocity is 1.04m/s with all the six zonal nodes being subjected to pressures between 3.82 bars to 4.8 bars which are within the required pressure range.

Table 4.40 presents hydraulic simulation optimal reservoir water release level results for the extended demand year 2027.

**Table 4. 40: Extended Year 2027 Optimal Reservoir Water Release Level.**

| Zonal Node ID | Pressure (m) | Valve ID | Type | Status | Setting (m) |
|---|---|---|---|---|---|
| RWRL (m) | 12.00 | 2 | TCV | Active | 55.58 |
| 35 | 41.54 | 34 | PRV | Active | 45.20 |
| 53 | 40.99 | 52 | PRV | Active | 50.00 |
| 81 | 34.41 | 77 | PRV | Active | 46.00 |
| 63 | 48.00 | 89 | PRV | Active | 48.00 |
| 102 | 38.14 | 101 | PRV | Active | 47.50 |
| 110 | 33.59 | 109 | PBV | Active | 9.00 |

The optimal reservoir water release level for extended year 2027 water demand period was simulated to be fifteen point four (15.4) meters but this could not be achieved, thus water release level was maintained at 12.0 meters which is the maximum height of water in the reservoir, with the main TCV no. 2 being set at 55.58 l/s. Pressure release valves (34, 52, 77, 89, 101) serving zonal node nos. 35, 53, 81, 63 and 102 have to be active and set at 4.52 bars, 5.0 bars, 4.6 bars, 4.8 bars and 4.75 bars respectively while PBV no. 109 serving zonal node no.110 need to be set at 9 meters. The overall flow velocity is

1.13m/s with zonal nodes 81 and 110 having pressures of 3.44 bars and 3.35 bars which are below the required minimum system pressures of 3.8 bars

Table 4.41 presents hydraulic simulation optimal reservoir water release level results for the extended demand year 2028.

**Table 4. 41: Extended Year 2028 Optimal Reservoir Water Release Level.**

| Zonal Node ID | Pressure (m) | Valve ID | Type | Status | Setting (m) |
|---|---|---|---|---|---|
| RWRL (m) | 12.00 | 2 | TCV | Active | 59.30 |
| 35 | 39.25 | 34 | PRV | Active | 45.20 |
| 53 | 38.38 | 52 | PRV | Active | 50.00 |
| 81 | 31.06 | 77 | PRV | Active | 46.00 |
| 63 | 44.78 | 89 | PRV | Active | 48.00 |
| 102 | 33.83 | 101 | PRV | Active | 47.50 |
| 110 | 29.34 | 109 | PBV | Active | 9.00 |

The optimal reservoir water release level for extended year 2028 water demand period was simulated to be sixteen point seven (16.7) meters, this could not be achieved, thus water release level was maintained at 12.0 meters which is the maximum height of water in the reservoir, with the main TCV no. 2 being set at 59.30 l/s. Pressure release valves (34, 52, 77, 89, 101) serving zonal node nos. 35, 53, 81, 63 and 102 have to be active and set at 4.52 bars, 5.0 bars, 4.6 bars, 4.8 bars and 4.75 bars respectively while PBV no. 109 serving zonal node no.110 need to be set at 9 meters. The overall flow velocity is 1.21m/s with zonal nodes 81, 102 and 110 having pressures of 3.10 bars, 3.38 bars and 2.93 bars respectively which are below the required minimum system pressures of 3.8 bars.

Table 4.42 presents hydraulic simulation optimal reservoir water release level results for the extended demand year 2029.

**Table 4. 42: Extended Year 2029 Optimal Reservoir Water Release Level.**

| Zonal Node ID | Pressure (m) | Valve ID | Type | Status | Setting (m) |
|---|---|---|---|---|---|
| RWRL (m) | 12.00 | 2 | TCV | Active | 61.82 |
| 35 | 37.60 | 34 | PRV | Active | 45.20 |
| 53 | 36.50 | 52 | PRV | Active | 50.00 |
| 81 | 28.66 | 77 | PRV | Active | 46.00 |
| 63 | 41.95 | 89 | PRV | Active | 48.00 |
| 102 | 30.76 | 101 | PRV | Active | 47.50 |
| 110 | 26.31 | 109 | PBV | Active | 9.00 |

For the extended year 2029 water demand period, the simulated optimal reservoir water release level is eighteen point six (18.6) meters but this could not be achieved, thus water release level was maintained at 12.0 meters which is the maximum height of water in the reservoir with all the control valves setting for year 2028 being maintained except TCV no.2 being adjusted to 61.82 l/s to accommodate the system water demand. The overall flow velocity is 1.26m/s with zonal nodes 35, 53, 81, 102 and 110 having pressures of 3.76 bars, 3.65 bars, 2.86 bars, 3.07 bars and 2.63 bars respectively which are below the required minimum system pressures of 3.8 bars.

Table 4.43 presents hydraulic simulation optimal reservoir water release level results for the extended demand year 2030.

**Table 4. 43: Extended Year 2030 Optimal Reservoir Water Release Level.**

| Zonal Node ID | Pressure (m) | Valve ID | Type | Status | Setting (m) |
|---|---|---|---|---|---|
| RWRL (m) | 12.00 | 2 | TCV | Active | 63.44 |
| 35 | 36.49 | 34 | PRV | Active | 45.20 |
| 53 | 35.23 | 52 | PRV | Active | 50.00 |
| 81 | 27.05 | 77 | PRV | Active | 46.00 |
| 63 | 40.04 | 89 | PRV | Active | 48.00 |
| 102 | 28.70 | 101 | PRV | Active | 47.50 |
| 110 | 24.28 | 109 | PBV | Active | 9.00 |

The optimal reservoir water release level for extended year 2030 water demand period was simulated to be twenty point seven (20.7) meters, this could not be achieved, thus water release level was maintained at 12.0 meters which is the maximum height of water in the reservoir, with all the control valves setting for year 2029 being maintained except TCV no.2 being adjusted to 63.44 l/s to accommodate the system water demand. The overall flow velocity is 1.29m/s with zonal nodes 35, 53, 81, 102 and 110 having pressures of 3.64 bars, 3.52 bars, 2.70 bars, 2.87 bars and 2.42 bars respectively which are below the required minimum system pressures of 3.8 bars.

### 4.4.4 System Water Demand and Optimal Reservoir Water Release Level

Table 4.44 and Table 4.45 are tabulations of the results for periodic system water demand and annual system water demand and their respective optimal reservoir water release levels while Figure 4.20 shows graphical representation of periodic water demand versus the respective optimal reservoir water release levels. From the results it can generally be observed that as the water demand increases, the optimal reservoir water release levels also increases.

**Table 4. 44: January 2017 to December 2020 Optimal Reservoir Water Release Levels.**

| Period | System Water Demand (l/s) | Optimal Water Release Level (m) |
|---|---|---|
| Jan 2017 - March 2017 | 29.37 | 7.7 |
| April 2017 - Sep 2017 | 30.09 | 8.0 |
| Oct 2017 - March 2018 | 31.32 | 8.0 |
| April 2018 - Sep 2018 | 32.40 | 8.0 |
| Oct 2018 - March 2019 | 33.96 | 8.2 |
| April 2019 - Sep 2019 | 35.09 | 8.2 |
| Oct 2019 - March 2020 | 36.22 | 8.5 |
| April 2020 - Sep 2020 | 36.25 | 8.5 |
| Oct 2020 - Dec 2020 | 37.11 | 8.7 |

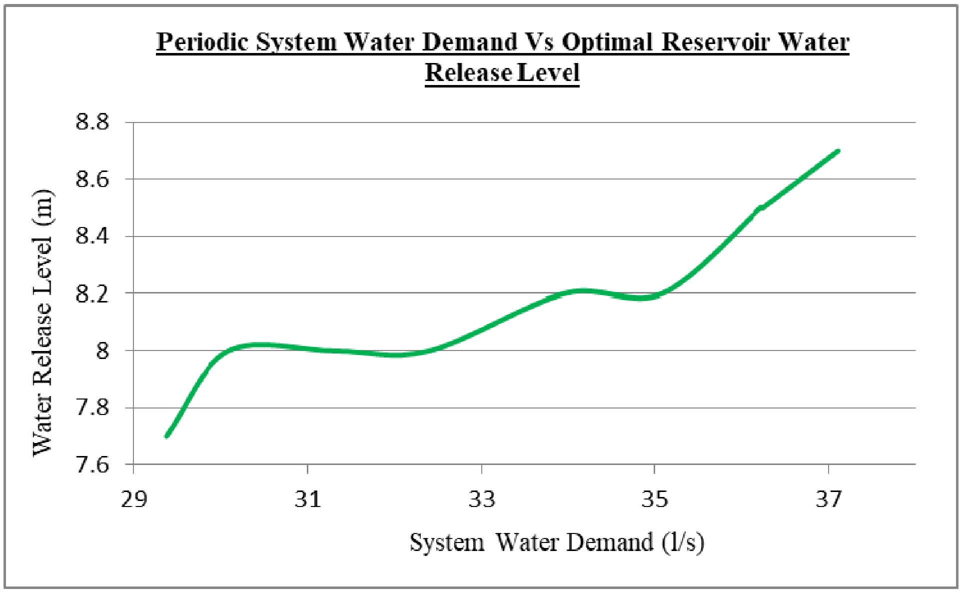


**Figure 4. 20: Periodic System Water Demand Versus Optimal Reservoir Water Release Level.**

Further analysis of the graph of periodic system water demand against periodic optimal reservoir water release level by drawing the best curve of fit using excel established that the general relationship between periodic system water demand and periodic optimal reservoir water release level is a polynomial function of order six defined by Equation 4.5 and figure 4.21.

$$y = -0.0005x^6 + 0.0964x^5 - 8.0484x^4 + 357.97x^3 - 8944.5x^2 + 11904x - 659369 \quad \text{Equation (4.5)}$$

Where;

y = Optimal reservoir water release level in meters.

x = System water demand in l/s.

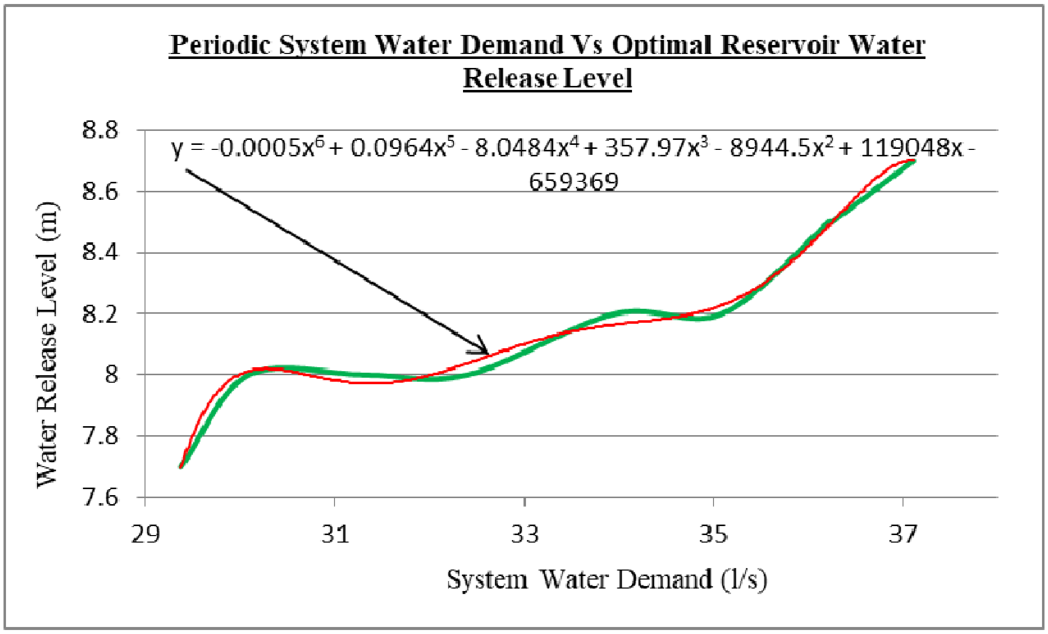


**Figure 4. 21: Trendline For Mathematical Relationship Between Periodic System Water Demand and Optimal Reservoir Water Release Level.**

**Table 4. 45: 2021 to 2030 Optimal Reservoir Water Release Levels**

| Year | System Water Demand (l/s) | Optimal Water Release Level (m) |
|---|---|---|
| 2021 | 38.39 | 8.70 |
| 2022 | 40.86 | 8.90 |
| 2023 | 44.25 | 9.00 |
| 2024 | 46.16 | 10.40 |
| 2025 | 47.96 | 11.20 |
| 2026 | 50.99 | 12.00 |
| 2027 | 55.58 | 15.40 |
| 2028 | 59.30 | 16.70 |
| 2029 | 61.82 | 18.60 |
| 2030 | 63.44 | 20.70 |

# CHAPTER FIVE: CONCLUSIONS AND RECOMMENDATIONS

## 5.1 Conclusions

This study sought to establish reservoir water release levels that would optimize pressure at the water pipeline distribution network zonal nodal points for the period 2017 to 2030. A water demand forecast of Kimilili Water supply scheme for the said period was to be carried out because of seasonal changes in water demand. Again a simulation of Kimilili water supply distribution network zonal nodal pressures to establish optimal reservoir water release levels that would maintain minimum allowable zonal nodal pressures for the period 2017 to 2030 was to be undertaken. This study established that:

The water demand for Kimilili water supply scheme was increasing with time, it ranged from 27.83 l/s to 36.26 l/s for January 2017 to December 2020 and from to 38.39 l/s to 63.44 l/s for 2021 to 2030 respectively as depicted by the prediction of MATLAB ANN model. It was established that the rains were usually high between October and March and low between April and September, consequently the water demand were low during the high rain seasons and vice versa, thus the water demand for the two rain periods varied with high values of water demand being witnessed in low rain season and low values of water demand being witnessed in the high rain season. The study further demonstrated that the general relationship between period (time) and water demand was a polynomial function of order six defined as $y = 9e\text{-}0x^6 - 1e\text{-}05x^5 + 0.0005x^4 - 0.0115x^3 + 0.1178x^2 + 0.1384x + 100.48$. Furthermore the general relationship between period and extended annual water demand was a polynomial function of order five defined as $y = -0.0021x^5 + 22x^4 - 8.7e{+}0.4x^3 + 1.8e{+}0.8x^2 - 1.8e{+}11x + 7.2e{+}13$.

System water losses reduced with increase in system water demand as demonstrated by the EPANET 2.0 hydraulic simulation model for Kimilili water supply scheme pipeline distribution network. When zonal nodal water demand increased the pressure for the respective zonal node reduced, consequently the total zonal node pressure reduction led to significant system pressure reduction hence reduction in the amount of water lost due to system pressures. The study also demonstrated that the general relationship between periodic system water demand and system water losses for Kimilili water supply scheme pipeline distribution network was an exponential function defined as $y = 256394e^{-7.296x}$. The study further established that the general relationship between periodic system water demand and percentage system water losses for Kimilili water supply scheme pipeline distribution network was a second order polynomial function defined as $y=1.8503x^2 - 21.882x + 88.808$.

The optimal reservoir water release level for Kimilili water supply scheme was on an upward trend with respect to time, demonstrated by the EPANET 2.0 hydraulic simulation model for Kimilili water supply scheme pipeline distribution network. The optimal reservoir water release levels were ranging from 7.7 meters to 8.7 meters with January to March 2017 period having 7.7 meters and October 2020 to December 2020 period having 8.7 meters. Likewise for the extended annual water demand period the optimal reservoir water release levels ranged from 8.7 meters to 12.0 meters with the year 2021 having 8.7 meters and the year 2026 having 12.0 meters. The study established that pressure management through optimization of reservoir water release levels for Kimilili Water Supply Scheme could be utilized up to the year 2026, beyond the year 2026 the optimized reservoir water release levels are 15.5 meters, 16.7 meters, 18.6 meters and 20.7 meters for the years 2027, 2028, 2029 and 2030 respectively, this cannot be practically achieved as the maximum water level in the reservoir that could be achieved is 12.0 meters.

Generally it was observed that as the water demand increased, the optimal reservoir water release levels also increased. The overall flow velocities in the transmission mains increased with time from 0.60m/s for January 2017 to March 2017 period to 0.76m/s for October 2020 to December 2020 period and extended to 1.04m/s for the year 2026, this was attributed to the respective increases in water demand with time. The study finally established that the general relationship between periodic system water demand and optimal reservoir water release level for Kimilili water supply scheme pipeline distribution network was a sixth order polynomial function defined by $y = -0.0005x^6 + 0.0964x^5 - 8.0484x^4 + 357.97x^3 - 8944.5x^2 + 11904x - 659369$. Thus given any water demand 'y' in l/s the optimal reservoir water release level 'x' in meters can be calculated.

## 5.2 Recommendations

Water demand for potable water will always increase with time due to growth in population regardless of the potable water supply infrastructure (systems) put in place. In order to reduce over production of potable water leading to high system water losses, this study recommends that Nzoia water services company should optimize on production at the existing Kimilili water supply scheme by practicing water demand management through systematically matching system water demand requirements and the respective optimal reservoir water release levels with the required control valve's settings as established by this study.

Hydraulic simulation of the EPANET 2.0 model has demonstrated that it is possible to reduce the system water losses by managing the pipeline network pressures using the optimal reservoir water release levels with the required control valve's settings as established by this study, without undertaking further water supply infrastructural expansions (investments). Consequently the study recommends that the established optimal reservoir water release levels with their respective system water demands to be adopted and implemented for management of pipeline network system pressures because the existing water supply scheme infrastructure has the potential of sufficiently serving all its network connections at the required pressures and flows up to the year 2026.

After 2026 the existing water supply infrastructure might be used rationing water supply to various zones depending on the pressure topologies. The long term solution will be to construct a higher and lager water reservoir at the treatment works and at the same time upgrade the distribution pipeline network.

### 5.2.1 Areas For Further Research

The study suggests that both long and short term model research should be done considering local utility changes in supply flows, financial policies and extending conservation plans.

The study further proposes that more research need to be carried out to determine Kimilili water supply scheme pipeline network system water demands from 2021 to the year 2030 considering the adoption of a new water tariff in March 2018 and the minor pipeline network extensions being undertaken by the County Government of Bungoma.

This will establish whether the existing system will be able to accommodate the water demand by then or if there will be need for expansion of the infrastructure.

### 5.3 Summary

It is evident that there is a link between the system water demand, the reservoir water release levels, system network pressures and the resulting system water losses. It is proposed that to sustain the zonal node pressures for the period 2017 to 2026 for Kimilili Water supply scheme pipeline distribution network, the optimal reservoir water release levels should range between 7.7m to 12.0 m. Implementing pressure management through optimization of reservoir water release levels for Kimilili Water Supply Scheme might only be applicable up to the year 2026, beyond which it might not be practicable.

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# APPENDICES

## APPENDIX I: RESEARCH CLEARANCE PERMIT

**CONDITIONS**

1. You must report to the County Commissioner and the County Education Officer of the area before embarking on your research. Failure to do that may lead to the cancellation of your permit.
2. Government Officer will not be interviewed without prior appointment.
3. No questionnaire will be used unless it has been approved.
4. Excavation, filming and collection of biological specimens are subject to further permission from the relevant Government Ministries.
5. You are required to submit at least two(2) hard copies and one (1) soft copy of your final report.
6. The Government of Kenya reserves the right to modify the conditions of this permit including its cancellation without notice

REPUBLIC OF KENYA

NACOSTI

National Commission for Science, Technology and Innovation

RESEACH CLEARANCE PERMIT

Serial No.A 13099

CONDITIONS: see back page

**THIS IS TO CERTIFY THAT:**
***MR. CELSUS MURENJEKHA SHILEHWA***
**of MASINDE MULIRO UNIVERISTY OF SCIENCE AND TECHNOLOGY, 1206-50205 WEBUYE,has been permitted to conduct research in *Bungoma County***

**on the topic: *OPTIMIZATION OF RESERVOIR PRESSURE FOR MANAGEMENT OF WATER LOSSES IN DISTRIBUTION NETWORK: CASE OF KIMILILI WATER SUPPLY SCHEME, KENYA.***

**for the period ending:**
***2nd February,2018***

**Permit No : NACOSTI/P/17/3261/15086**
**Date Of Issue : 9th March,2017**
**Fee Recieved :ksh1000**

...............................
***Applicant's Signature***

FOR

...............................
***Director General***
***National Commission for Science, Technology & Innovation***

# APPENDIX II: RESEARCH AUTHORIZATION

**NATIONAL COMMISSION FOR SCIENCE, TECHNOLOGY AND INNOVATION**

Telephone:+254-20-2213471,
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Uhuru Highway
P.O. Box 30623-00100
NAIROBI-KENYA

Ref: No.
**NACOSTI/P/17/3261/15086**

Date:
**9th March, 2017**

Celsus Murenjekha Shilehwa
Masinde Muliro University of
Science and Technology
P.O. Box 190-50100
**KAKAMEGA.**

**RE: RESEARCH AUTHORIZATION**

Following your application for authority to carry out research on ***"Optimization of reservoir pressure for management of water losses in distribution network: Case of Kimilili Water Supply Scheme, Kenya,"*** I am pleased to inform you that you have been authorized to undertake research in **Bungoma County** for the period ending **2nd February, 2018.**

You are advised to report to **the County Commissioner and the County Director of Education, Bungoma County** before embarking on the research project.

On completion of the research, you are expected to submit **two hard copies and one soft copy in pdf** of the research report/thesis to our office.

**DR. STEPHEN K. KIBIRU, PhD.**
**FOR: DIRECTOR-GENERAL/CEO**

COUNTY COMMISSIONER
BUNGOMA

Copy to:

The County Commissioner
Bungoma County.

COUNTY DIRECTOR OF EDUCATION
BUNGOMA

The County Director of Education
Bungoma County.



# APPENDIX III: TURNITIN DIGITAL RECEIPT

turnitin

# Digital Receipt

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The first page of your submissions is displayed below.

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| Submission title: | OPTIMIZATION OF RESERVOIR PR.. |
| File name: | MSc_Thesis_Report_Pressure_Op... |
| File size: | 29.06M |
| Page count: | 126 |
| Word count: | 29,095 |
| Character count: | 154,550 |
| Submission date: | 03-May-2019 04:58PM (UTC+0200) |
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OPTIMIZATION OF RESERVOIR PRESSURE FOR MANAGEMENT OF WATER LOSSES IN DISTRIBUTION NETWORK: CASE OF KIMILILI WATER SUPPLY SCHEME, KENYA

Celsus Murenjekha Shilehwa

A thesis submitted in partial fulfillment of the requirement for the award of degree of Master of Science in Water Resources Engineering of Masinde Muliro University of Science and Technology

May, 2019

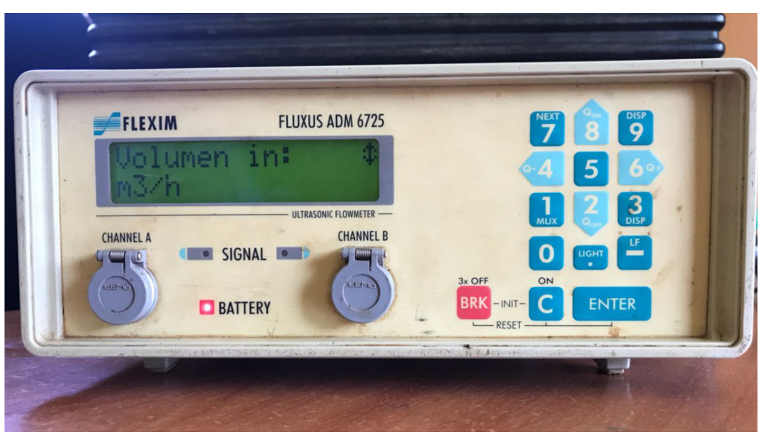
## APPENDIX IV: DATA COLLECTION INSTRUMENTS


**GARMIN GPSmap 60CSx.**


**GARMIN etrex 30x.**

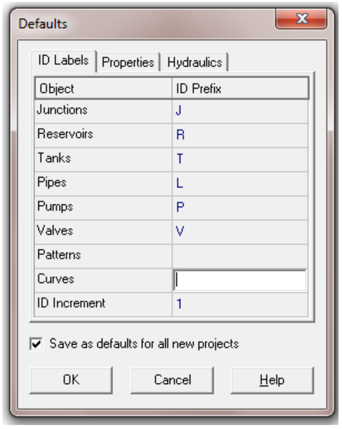

**TECHNOLOG Cello GSM Data Logger.**

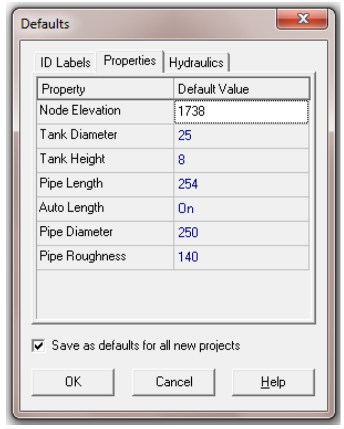

**KELLER IM Manometer.**

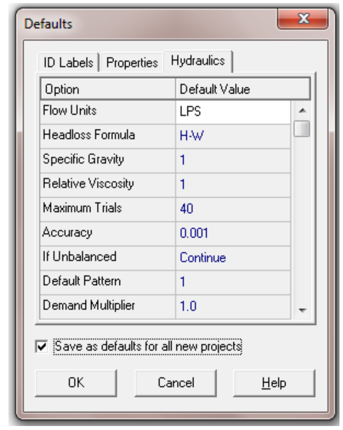


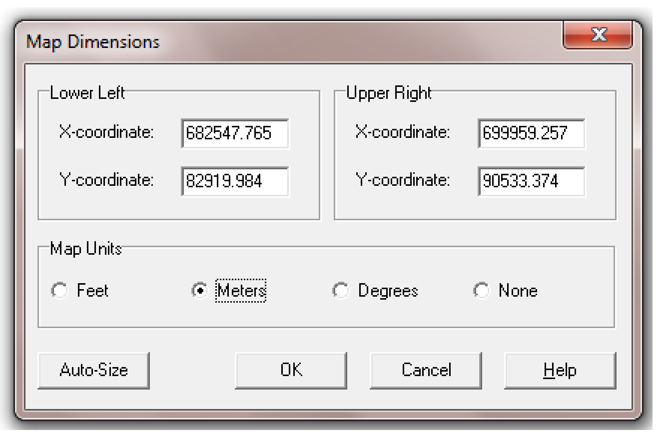

**A Clamp on Flexim Fluxus ADM 6725 Ultrasonic Flow Meter.**

# APPENDIX VI: EPANET 2.0 DEFAULT VALUE WINDOWS



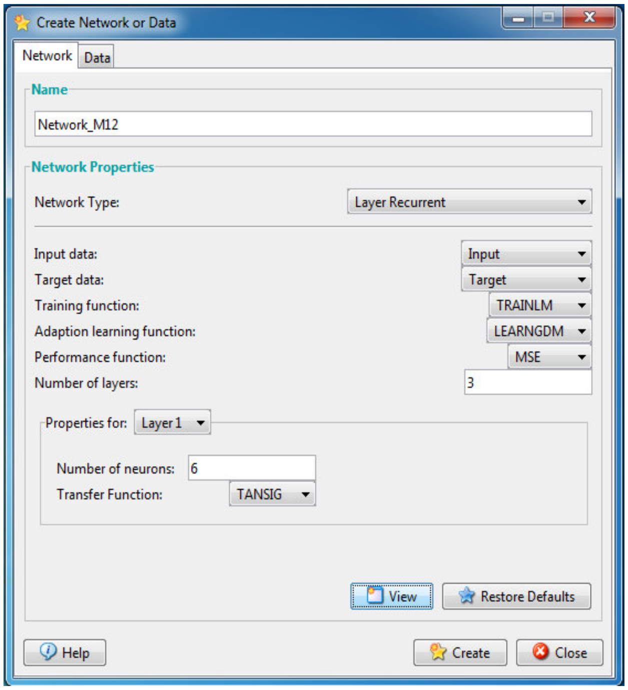



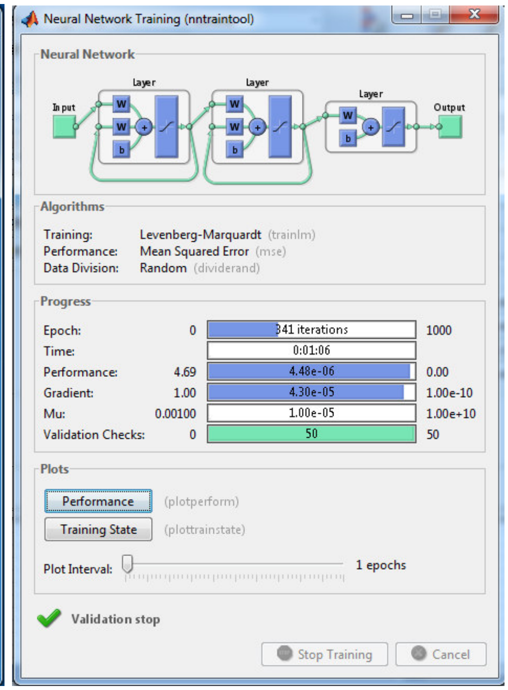



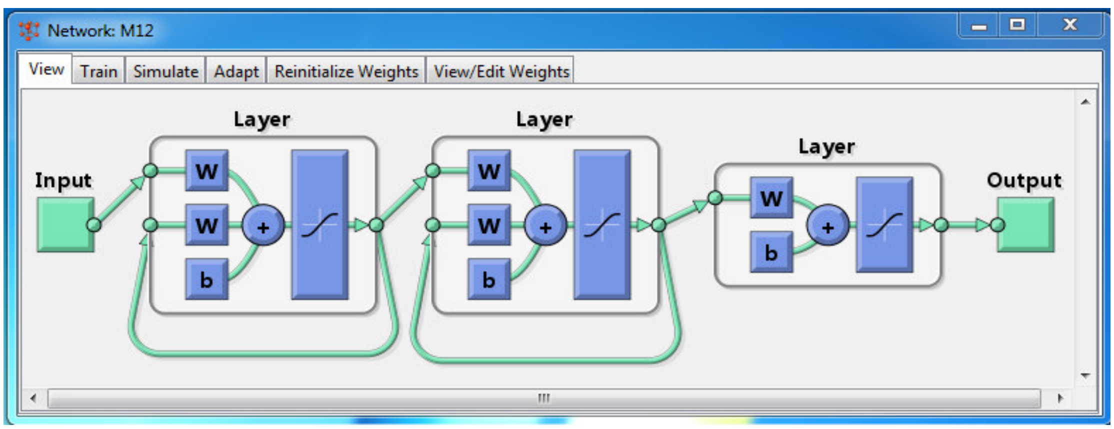



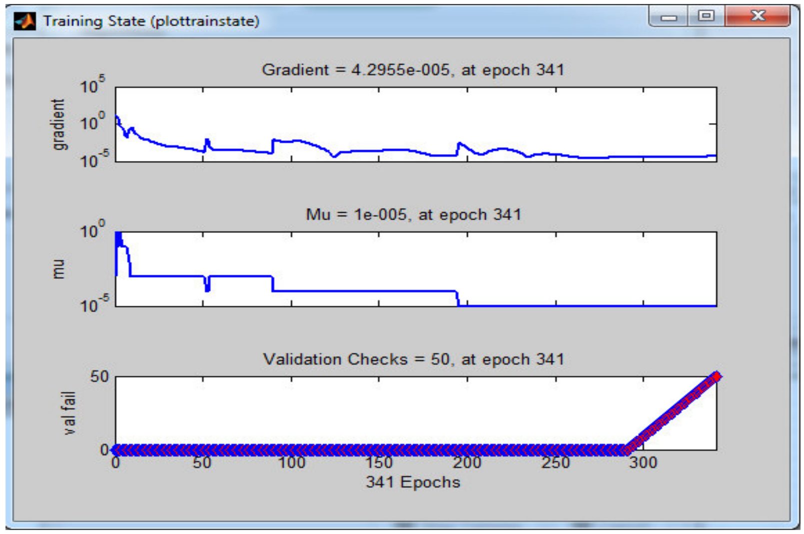

Georeferencing

# APPENDIX VII: ANN MODEL DEVELOPMENT PROPERTIES




Training State (plottrainstate)
Gradient = 4.2955e-005, at epoch 341
gradient
Mu = 1e-005, at epoch 341
mu
Validation Checks = 50, at epoch 341
val fail
341 Epochs