SIMULATION OF FLOW, HEADLOSS AND CHLORINE DECAY USING EPANET MODEL FOR MARALAL WATER DISTRIBUTION NETWORK IN SAMBURU COUNTY, KENYA

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DECLARATION AND RECOMMENDATION

Declaration

This thesis is my original work and has not been presented in this university or any other for award of a degree.

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Recommendation

This thesis has been submitted with our approval as University supervisors.

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DEDICATION

To the Eternal and Blessed God whose power and deity we have life. Secondly, to the residents of Maralal town, the Samburu County headquarter, who are the main water consumers of Maralal Water Supply whose un-intermittent water provision I treasure.

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ABSTRACT

Majority of people in developing countries do not have access to clean and potable water due to inadequate supply and distribution system challenges. While the rationale of water distribution systems is to deliver to each consumer safe drinking water that is adequate in quality and quantity at an acceptable delivery pressure, this has been a major drawback for many distribution networks. In addition, the design spans for many urban and peri-urban water distribution networks managed by the Water Service Providers (WSPs) are being exceeded without augmentation. Maralal water distribution network is one of such distribution systems with poor system performance that has been the main contributor of high Non-Revenue Water (NRW). This coupled with significant mismatch between water supply and water demand makes Maralal Water and Sanitation Company to resort to hedging/intermittent flow leading to water rationing. One of the ways of predicting the flow dynamics within the distribution system is the use of hydraulic simulation models. This study therefore applied the Environmental Protection Agency Network (EPANET) simulation model to predict the dynamic state of the hydraulics and water quality status for Maralal water distribution system operating over an extended period of time. The general objective was to simulate water flow for Maralal water distribution system using the EPANET model for efficient planning, operation and maintenance protocol for the system. The parameters simulated were flow, headloss and residual chlorine. The model flow calibration results from four statistical criteria; Nash-Sutcliffe model efficiency coefficient (E), Sum of Squares Error (SSE), Percentage Bias (PB) and Root Mean Square Error (RMSS) of 0.99, 0.01, 0.05 and 0.03 respectively show that the model performed within acceptable range of the selected statistical criteria. The model was validated with Day 1 and Day 3 flow rate data for PIPE 1 in County zone. The findings of this study were: The roughness coefficients for a water distribution network that contribute to erratic pressure-dependent flows could be determined at any time using the regression analysis of the measured head loss and flow rate, EPANET model predicted the steady and dynamic hydraulic parameters for the current and future water distribution systems and Chlorine decay with respect to pipe diameter impacts on hydraulic performance and quality of water in a distribution network. The results from this study would assist water service providers and managers to make informed decisions in relation to water distribution system planning, operation and maintenance to achieve the desired water demands now and in the future.

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LIST OF ABBREVIATIONS

EPANET	Environmental Protection Agency Network
EVT	Extreme Value Theory
GTA	Generic Trace Analysis
IWS	Intermittent Water Supply
MAWASCO	Maralal Water and Sanitation Company
NWSB	Northern Water Service Board
UFW	Un-accounted for Water
UNICEF	United Nations International Children Education Fund
WASREB	Water Services Regulatory Board
WSB	Water Service Board
WSP	Water Service Provider

CHAPTER ONE

INTRODUCTION

1.1 Background Information

The purpose of any water distribution system is to deliver to each consumer safe drinking water that is adequate in quantity, delivery pressure and acceptable in terms of taste, odour and appearance. However, continued population growth has placed increasing demand upon existing water distribution systems. This growth has necessitated the need to analyze existing and design new water distribution systems. In addition, recent concern and awareness about the safety of drinking water has raised other concerns on the quality of water delivered in the existing and proposed municipal or city water distribution systems.

Water distribution networks present complex systems that include different types of pipes and sizes, diverse types of valves, tanks and pumps. These networks require significant investments for construction, operation and maintenance. In this regard, the awareness of all hydraulic parameters in a water distribution system is an absolute prerequisite for rational planning of new networks and upgrading of existing system elements. If the system elements, their functions and hydraulic parameters are not known, numerous problems are likely to occur at some point in time during the operation of the system. For water distribution networks, the challenges are mainly due to low or high pressures in certain parts of the system, occurrence of system defects, such as leakage and increased energy consumption (Muranho *et al.,* 2014).

Water utility enterprises in developed countries have already started researching on strategic solutions for water distribution systems rationalization, and water consumption optimization by use of simulation models (Araceli *et al.,* 2020). In order to meet regulatory requirements and customer expectations, Water Service Providers are globally faced with the challenge of understanding their water distribution systems. One of the ways to understand the water distribution systems could be achieved through the analysis of the flow dynamics in the distribution systems. Simulation of the flow can offer alternative options in addressing the water distribution systems' challenges. Models can be used to predict the dynamic state hydraulics and water quality behaviour for a drinking water distribution system operating over an extended period. The models can also be used as tools to assist in the planning, operation and maintenance decision making.

This research will contribute to water availability to communities in Maralal in line with the United Nations Sustainable Development Goal (SDG 6) whose official wording is 'Ensure availability and sustainable management of water and sanitation for all'. SDG 6 is one of the 17 Sustainable Development Goals established by the United Nations General Assembly in 2015.

1.2 Statement of the Problem

Over the years, the actual pipe roughness factor within a chlorine disinfected water distribution networks has rarely been periodically determined as one of the factors to establish and predict the hydraulic performance of the system. This has contributed to the design span of many urban and peri-urban water utilities in Kenya being exceeded without informed augmentation plans. Roughness within aged water distribution networks causes friction which contribute to system head loss that continues to limit the systems performance. In addition to the roughness effect on the system performance, chlorine decay during water distribution within a network over time causes biofilm formation. In addition, it causes corrosion and wear of pipe walls. The decay also affects water distribution, as higher levels of residual chlorine beyond regulatory requirements are undesirable. This not only causes damage of the network elements but also results to consumer complaints in regard to the quality of water in terms of odour and taste. For instance, levels of residual chlorine below the regulatory requirements of 0.2mg/l have been found in many consumer points in Maralal water distribution network. This poses a challenge to the management of MAWASCO in regard to the formulation of optimum chlorine dosing flow rate at the treatment works. Variation of pipe roughness affects the energy dissipation (headloss) in a pipeline hence limited system performance. Poor system performance of Maralal water distribution network is the main contributor of high Non-Revenue Water (NRW). This coupled with significant mismatch between water supply and water demand makes MAWASCO to resort to hedging, a phenomenon commonly referred to as water rationing.

1.3 Objectives

The objectives of this research were categorized into broad and specific objectives. The Broad objective was the goal which established the scope of the study. The specific objectives comprehended on the problem to be addressed by the study which produced the data requisite data.

1.3.1 Broad Objective

The broad objective was to simulate water flow for Maralal water distribution system using the Environmental Protection Agency Network (EPANET) model for efficient planning, operation and maintenance protocol for the system.

1.3.2 Specific Objectives

The specific objectives were to:

i. Determine the current roughness factors for each pipe within the network.
ii. To simulate and predict the steady and dynamic states hydraulic parameters using EPANET model; and
iii. Determine the impact of chlorine decay on the system hydraulic performance.

1.4 Research Questions

i. What are the current roughness coefficients for Maralal water distribution network that contribute to erratic pressure-dependent flows?
ii. How can the EPANET model predict the steady and dynamic hydraulic parameters for the current and future Maralal water distribution systems?
iii. How does chlorine decay with respect to pipe diameter impact on hydraulic performance and quality of water in a distribution network?

1.5 Significance of the Study

Although data is required to inform water consumers whenever their water demands are not met, most water providers do not have reliable methods of addressing such occurrences especially when the measuring devices are faulty or sometimes non-existence. One way of providing such supportive data is through the application of hydraulic simulation.

This study therefore sought to apply the EPANET model to provide water service providers in Maralal with reliable information and data to help in evaluating the Maralal system hydraulic parameters. This would help in timely detection of system failure and designing economical distribution systems that meet both regulatory and consumer requirements. Results of this study are useful to the water resources managers in enhancing water service delivery. In addition, the results could assist in equipping the Water Service Providers (WSPs) with the necessary information to make optimal decisions in relation to system planning, operation and maintenance to achieve the desired service delivery now and in the future.

CHAPTER TWO
LITERATURE REVIEW

2.1 Water Distribution Networks

Water is undoubtedly one of the essential commodities, with no other alternative, every living creature requires for survival (Alkali *et al.*, 2017). Although this study concurs with the observations by these authors, it should be noted that providing adequate amount of clean and potable water has been one of the most challenging issues in human history. In developing countries, majority of the population do not have access to clean water due to inadequate supply and distribution system challenges (Adeniran & Bamiro, 2010). The gap in Adeniran and Bamiro (2010) statement is that water distribution system challenges are diverse and need to be approached as specific aspects in order to ensure efficiency of the system. With reducing and unpredictable rainfall pattern and lack of perennial water sources, there is need to design a water distribution network with minimal hydraulic losses (Jagrat & Hari, 2019). This study agrees with Jagrat and Hari (2019) as hydraulic losses limits a water distribution system performance and contribute to limited water supply to communities.

To many hydraulic engineers, it is common knowledge that the roughness factor in pipes change over time. According to Ildeberto *et al*. (2021), pipes that have been in operation for a considerable time, two physical settings change drastically compared with the initial design: the roughness and the inner diameter. The same authors added that the variation of these parameters affects energy dissipation (headloss) in a pipeline which this study concurs with them. In particular, Feinauer *et al.* (2008) found out that changes in roughness factor could cause the head loss to increase by nearly 45%. However, the effect of this phenomenon on the hydraulics and the quality of water flowing through a pipe network is not well documented in many water distribution systems and this was identified as one of the gaps that the current study addressed with a view to addressing the challenges facing the Maralal water distribution system.

In some cases, the estimated roughness of old pipes, using optimization procedures based on measurements in operational water distribution system, can be quite demanding (Ivar & Anatoli, 2015). However, according to Naser and Mohammad (2014), hydraulic network analysis provides an option to understanding the malfunctioning of a water distribution system. The statement by Naser and Mohammad (2014) is therefore applicable to this research objective as change in roughness factor over time contributes to the malfunctioning of a distribution system.

Hydraulic network analysis is the process of using a water distribution system computer model to analyze the system performance capabilities and to define the requirements necessary to meet its design standards for pressure and flow. Applications of hydraulic network analysis generally fall into three categories: planning, design and operations.

A primary planning application of network analysis includes; scheduling, staging, sizing, preliminary routing and locating of future facilities. This master planning of communities, counties, and municipalities requires the expertise of city planners and civil engineers, who must consider many factors, such as location, current demand, future growth, leakage, pressure, pipe size, pressure loss and firefighting flows (Anisha *et al.*, 2016). Network- analysis- design applications considers all the above factors and in this respect, this study agrees with the cited authors.

Pipelines, pressure-regulating valves and ground Tanks can be sized using pressure and flow calculations resulting from hydraulic simulation**.** This study supports Vasan and Simonovic (2010) who suggested the design and simulation of the water distribution network by modeling, analyzing and performance evaluation through scenario investigation of the physical and hydraulic parameters.

The development of operating strategies, operator training and system maintenance are applications supported by simulation system operations. However, water supply deficiency is the major drawback in urban area since the water distribution system is an intermittent system. Under this scheme water is distributed to the residents intermittently for few hours in a week. Due to the intermittent water supply, most of the time the pipelines in the distribution network are either empty or partially filled. Hydraulic simulation could also be used to develop operational strategies based on energy management guidelines and restrictions for more efficient system operations.

Simulation and network analysis are some of the strategies used in training personnel involved in the operation of distribution systems. The simulation approach is preferred since it enables the distribution system operators to experiment with the model to the system performance under specified operating conditions.

Also, water service providers are focusing on the behaviour and transport of chemical species in a water distribution system using hydraulic models. Beginning in the mid-eighties, advancements in computer technology have allowed for the addition of water quality assessment to hydraulic models. This has been motivated by the recognition that water quality can greatly change from the water treatment plant, through the distribution system and to the consumer. With the advancement in dynamic hydraulic simulations, the long-term simulation of water quality within a distribution system became possible. Most versions of water distribution models contain a water quality simulation package in addition to hydraulic simulation. With the capability of water quality simulation, analyses have been made that has improved the understanding of reaction and transport of different chemical constituents.

2.2 Network Hydraulic Simulation

Water supply planners and engineers use a range of models to support in decision-making. Network hydraulic simulation is a key activity to gain an understanding of how the water supply system operates under various demand/flow scenarios, now and into the future. It is also used to assess the performance of the water supply system in the event of various failures (e.g. critical asset failure or overflows). In addition, the network hydraulic simulation is useful in assessing the impacts of proposed operational modifications, augmentations or renewals. The Network hydraulic simulation becomes handy in reviewing the impacts of proposed developments and also providing the supporting information for a planning study.

Successful network simulation requires the investment of time by experienced staff to interpret the results of the simulation. Operational staff should therefore be involved in the construction and analysis of the network model.

For larger service providers with high populations growth rates, network simulation is an ongoing process being undertaken by skilled in-house staff and/or external consultants. For smaller service providers with low growth rate populations, network simulation is being undertaken intermittently to support specific planning studies or to identify the cause of operational problems.

2.2.1 Network Solution Methods

Network solution methods have evolved from applications where networks were solved by hand calculations to solutions supported by computer software. For instance, Adeleke and Olawale (2013) developed a computer program of pipe network analysis using Java programming language for the Hardy Cross method to study some existing pipe network in Osun State to evaluate their suitability towards sustainable resource planning. The Hardy Cross method is an adaptation of the Moment distribution method, which was also developed by Hardy Cross as a way to determine the moments in indeterminate structures. The introduction of the Hardy Cross method for analyzing pipe flow networks revolutionized municipal water supply design. Before the method was introduced, solving complex pipe systems for distribution was extremely difficult due to the nonlinear relationship between the head loss and flow rate. The method was later improved through use of computer solving algorithms employing Newton-Raphson method or other solution methods that replaced the need to solve nonlinear systems of equations manually. Pipe network analysis of water distribution systems has evolved from a time consuming process done infrequently to a quick and easy process done regularly on systems of all sizes. Within a network with multiple loops, the Hardy-Cross method determines a loop equation for each loop and solves one loop at a time. This method requires a flow balance before the first iteration. The initial flow directions are normally assumed.

With the advancement of computing technology with high-speed computers available, it is now possible to develop algorithms that can solve the entire system network. One of the most commonly applied algorithms is called the gradient method which was reported by Aderian and Oyelowo (2013). This method allows a modeler to analyze large networks by solving a system of partly linear and non-linear equations that express the balance of mass and energy. The method requires an initial guess for the pipe flows and nodal heads. Unlike the Hardy-Cross solution, the gradient method does not require a flow balance. An advantage of this approach is that all pipe flows and nodal hydraulic heads are solved in each iteration. This allows the gradient method to converge on a solution in little iteration than other methods such as Hardy-Cross.

2.2.2 Determination of Friction Losses

There are a number of head loss equations that have been developed to determine frictional losses through a pipe. The three most common equations are the Manning, Hazen-Williams and Darcy-

Weisbach equations. The Chezy-Manning equation is more typically used for open channel flow and is dependent on the pipe length and diameter, flow and the roughness coefficient. The Chezy-Manning equation is given as:

$$h_{L}= \frac{C_f L(nQ)^2}{D^{5.33}} \tag{2.1}$$

Where:

h_L = head loss (m),

n = Manning roughness coefficient (dimensionless),

C_f = Unit conversion factor (SI = 10.675),

L = Pipe length (m),

D = Pipe diameter (mm),

Q = Pipe Flow (m^3/s).

The Hazen-Williams equation has been used mostly in North America and is distinctive in the use of a C-factor. The C-factor is used to describe the carrying capacity of a pipe. High and low C-factors represent smooth and rough pipes respectively. The Hazen-Williams equation (Walski *et al*., 2003) for the computation of head loss is given as:

$$h_{L}= \frac{C_f L}{C^{1.852} D^{4.87}} Q^{1.852} \tag{2.2}$$

Where:

C = Hazen-Williams C-factor.

The Darcy-Weisbach equation was developed using dimensional analysis. This expression uses many of the same variables as the Hazen-Williams equation, but rather than using a C-factor, it uses a friction factor, f. The Darcy-Weisbach equation for the computation of head loss is given as:

$$h_{L}= \frac{8fLQ^2}{gD^5\pi^2} \tag{2.3}$$

Where:

f = Darcy-Weisbach friction factor,

g = Gravitational acceleration constant (m/s^2).

The Hazen-Williams formula is the most commonly used head loss formula in the United States. 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. Each formula uses the following equation to compute head loss between the start and end node of the pipe:

$$h_L = Aq^B \tag{2.4}$$

Where:

h_L = head loss (m),

q = flow rate (m^3/s),

A = resistance coefficient,

B = flow exponent.

Table 2.1 lists expressions for the resistance coefficient and values for the flow exponent for each of the formula. Each formula uses a different pipe roughness coefficient that must be determined empirically.

Table 2.1: Pipe Head Loss Equation for Full Flow

Formula	Resistance Coefficient (A)	Flow Exponent (B)
Hazen-Williams	$10.675\ C^{-1.852}\ d^{-4.871}\ L$	1.852
Darcy-Weisbach	$0.0827\ f(\varepsilon,d,q) d^{-5} L$	2
Chezy-Manning	$4.66\ n^2\ d^{-5.33}\ L$	2

Where: C = Hazen-Williams roughness coefficient

ε = Darcy-Weisbach roughness coefficient

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

n = Manning roughness coefficient

d = pipe diameter (mm)

L = pipe length (m)

q = flow rate (m^3/s)

Table 2.2 presents general ranges of these coefficients for different types of new pipe materials. Darcy-Weisbach formula uses different methods to compute the friction factor f depending on the flow regime. Hagen–Poiseuille formula is used for laminar flow (Re < 2,000). On the other hand, the Swamee and Jain approximation to the Colebrook-White equation is used for fully turbulent flow (Re > 4,000) and the cubic interpolation from the Moody Diagram is used for transitional flow (2,000 < Re < 4,000).

Table 2.2: Roughness Coefficients for New Pipes

Material value (n)	**Hazen-Williams C (no units)**	**Darcy-Weisbach ($m \times 10^{-3}$)**	**Chezy-Manning's**
Cast Iron	130 – 140	0.26	0.012 - 0.015
Concrete	120 – 140	0.305 – 3.05	0.012 - 0.017
Galvanized Iron	120	0.15	0.015 - 0.017
Plastic	140 – 150	0.0015	0.011 - 0.015
Steel	140 – 150	0.046	0.015 - 0.017
Vitrified Clay	110	-	0.013 - 0.015

The Hazen-Williams equation is mainly used for head loss calculations over Darcy-Weisbach or Chezy-Manning equations. The Chezy-Manning equation is mainly used for open channel flows, and even though it is sometimes used for pressurized flow, it is not the most accurate method. Darcy-Weisbach is more accurate. However, Hazen-Williams is the most commonly used and it has been found to cause fewer computational problems.

2.3 Water Quality Simulation

Water quality simulation uses the flow results from the hydraulic calculations and the concepts of reaction kinetics to track the concentration of a substance in advective transport through the system (Alemtsehay & Tiku, 2017). Though the augments of this writer applies to any substance carried within a fluid in a pipe network, this study agrees with the fact that hydraulic calibration is a prerequisite to chlorine decay simulation. Advective transport is the bulk movement of chemicals with the carrier fluid. The transport process may occur while the substance is reacting (either growing or decaying) at some given rate. The equation for computing the rate of change of concentration for a chemical is given as:

$$\frac{dC_i}{dt} = -u_i \frac{dC_i}{dx} + rC_i \tag{2.5}$$

Where:

C_i = concentration in pipe *i* as a function of distance *x* and time *t*,

u_i = flow velocity in pipe *i*,

r = rate of reaction.

At junctions receiving inflow from two or more pipes, the mixing of fluid is taken to be complete and instantaneous. Thus the concentration of a substance in water leaving the junction is simply the flow-weighted sum of the concentrations from the inflowing pipes. For a specific node k the equation is given as:

$$\frac{\mathrm{Ci}}{\mathrm{x}} = 0 = \frac{\left(\Sigma \mathrm{Q\ j} \mathrm{C}\frac{\mathrm{j}}{\mathrm{x}} = \mathrm{L} + \mathrm{Qk,ext\ Ck,ext}\right)}{(\Sigma \mathrm{Q\ j} + \mathrm{Qk,ext})} \tag{2.6}$$

Where:

i = link with flow from node k,

Ci/x=0 = concentration at the start of link I,

C j/x=L = concentration at distance L in link j,

L = length of link j,

Q j = flow in link j,

Q k,ext = external source flow entering the network at node k,

Ck,ext = concentration of the external flow entering at node k (Viessman &Hammer, 1998).

2.4 Network Simulation Softwares

Comprehensive hydraulic simulation software applications in water and sanitation engineering consultancy practice are much essential to simplify several single line or small tree distribution iteration processes used in the olden days. Several hydraulic models are available for hydrology, water supply, storm water management and watershed management.

Recently, several computer programs running on personal computers, such as Environmental Protection Agency Network (EPANET), Water Distribution Software (WADISO), Distribution Engineering Workstation (DEW), and Water Computer Aided Design (WATERCAD) have been developed and made available.

2.4.1 WADISO Software

Water Distribution Software (WADISO), is a comprehensive computer program for the analysis and optimal design of water distribution networks. The software has three modules namely: Steady State Analysis Module; Extended Period Simulation Module and Optimization Module.

The Steady State Analysis Module is the basic module which allows for the input and editing of system data and parameters, and which calculates the flow and pressure distribution in the system under specific "snapshot" steady state conditions.

The Extended Period Simulation Module allows for modelling of diurnal fluctuations in water demand, and controls of pumps, valves, etc. in order to monitor system performance (e.g. tank level fluctuations, pumping cycles, pressure variations) over an extended period of time.

Optimization Module is a unique module that allows for the determination of future improvement needs, with the objective being to minimize capital expenditure and present worth of operational costs, while adhering to specified operational criteria. The cost trade-off between pipes and pumping costs, and pipes and storage cost are taken into account for the optimization. The software has no water quality analysis capabilities therefore it is fully applicable for this study.

2.4.2 DEW Software

DEW has the ability to simulate water system hydraulics by the use of the Hazen-Williams, Darcy-Weisbach, or Manning headloss equations. The equations for each method are programmed in DEW and the use of any of the methods is user selectable. The manner in which each equation accounts for pipe roughness is slightly different. The C-factor for the Hazen-Williams equation is obtained by DEW and is determined by the material of the pipe. The Darcy-Weisbach friction factor, f, is a function of the Reynolds number and relative roughness (roughness coefficient divided by the diameter of a pipe) and is determined using the

Swamee-Jain equation. Currently in DEW software, the friction factor is embedded directly into the source code, but will eventually be determined based on pipe material and flow conditions. The Manning roughness, *n*, is a function of the Reynolds number and the friction factor. This value is also directly embedded into the source code and not determined for each respective pipe. It is important to note that the DEW model will eventually account for the effects of pipe age on pipe roughness values (i.e. Hazen-Williams C-factor).

The disadvantage aspect of the hydraulic solution in DEW is the use of cotrees. A cotree is the result of forming a closed loop within a network. DEW depends on the pressure difference across a cotree for determining convergence. Convergence occurs in DEW when the pressure difference across a cotree is less than a set tolerance. When a system is designed in DEW the first action carried out by the model is determining the feeder path for each component within the system. This is time consuming especially when the network contains several loops. Therefore, the software is fully applicable to this study as the study network has no closed loop.

2.4.3 WATERCAD Software

Water Computer Aided Design (WATERCAD), is a complete geographic information management system for a water utility in a cost-effective package that saves money each time it is used. It determines flow requirements; calibrate large distribution networks and more so WATERCAD is powerful hydraulic analysis tools. It enables engineers and decision makers to analyze and manage distribution networks with unprecedented accuracy and efficiency. However, it is a sophisticated tool as it requires model building with geospatial modules and tools like LoadBuilder and TRex, fireflow analysis and scenario management. The software cannot be useful for this study as it only performs hydraulic analysis but not water quality simulation.

2.4.4 EPANET 2.0 Software

The Environmental Protection Agency Network (EPANET) is a computer program that performs extended period simulation of hydraulic and water quality behaviour within pressurized pipe networks. The flow network consists of pipes, nodes (pipe junctions), pumps, valves and storage tanks or Tanks. This software is available as free downloads and is very versatile for reasonable size of representative area and application. It has inbuilt drawing modules and data input modules.

In the flow network system, the EPANET is used to track the flow of water in each pipe, the pressure at each node, the head of water in each tank, and the concentration of chemical species throughout the network during a simulation period (Manoj *et al.,* 2018). In its design, EPANET can be a useful research tool that will enable water Engineers to improve their understanding of the movement and the constituents of drinking water within a given distribution system.

EPANET applications in solving and/or optimizing water distribution network problems have been reported by Abubakar and Sagar (2013); Adeniran and Oyelowo (2013); Fabunmi (2010) and Guidolin *et al.* (2010). The EPANET model can help assess alternative management strategies for improving water quality throughout a given water distribution system. These management strategies include: altering source utilization within multiple source systems; altering pumping and tank filling/emptying schedules; use of satellite treatment, such as re-chlorination at storage tanks and targeted pipe cleaning and replacement. Running under Windows, the EPANET model provides an integrated environment for editing network input data, running hydraulic and water quality simulations, and viewing the results in a variety of formats. These include color-coded network maps, data tables, time series graphs and contour plots.

The EPANET models a water distribution system acts as a collection of links connected to nodes. While the links represent pipes, pumps, and control valves, the nodes represent junctions, tanks, and Tanks. The model can be used for any size of the urban conglomerate. For instance, SAVEETHA University water distribution network has been designed using EPANET 2.0 (Manoj *et al.,* 2018). For simple applications up to 50,000 people, a water distribution network can be developed using inbuilt drawing tools and actual site conditions in can be simulated from a single EPANET diagram. Any large integrated urban water supply system is nothing but an integration of several subsystems.

Normally water supply distribution systems can be easily broken down in to sub-systems of less than 50,000 people, either from central overhead tank or local distribution tank or valve. For extension of existing water supply, EPANET can help in sizing the pipeline diameters based on the available head at the supply point to meet the quantity requirements as per the housing pattern. In addition, EPANET 2.0 can be a very useful tool in energy audit studies through simulation and analyzes of the existing water supply and distribution systems for their improvement, revamping and refurbishing.

2.4.5 EPANET Simulation Capabilities

The EPANET has the capability to model both the system hydraulics and the water quality. Full-featured and accurate hydraulic simulation is a prerequisite for doing effective water quality simulation. EPANET contains a state-of-the-art hydraulic analysis engine that includes the following capabilities: It tracks; the flow of water in each pipe, the pressure at each node, the height of the water in each tank, the type of chemical concentration throughout the network during a simulation period, water age, source, and tracing (Anisha *et al.*, 2016).

EPANET also uses zero or first order kinetics to model reactions at the pipe wall; allows growth or decay reactions to proceed up to a limiting concentration; employs global reaction rate coefficients that can be modified on a pipe-by-pipe basis; allows wall reaction rate coefficients that was correlated to pipe roughness; allows for time-varying concentration or mass inputs at any location in the network and models storage tanks as being either complete mix, plug flow, or two-compartment reactors.

2.4.6 EPANET Hydraulic Simulation Method

The method for solving flow continuity and head loss equations in EPANET is known as a hybrid node-loop approach. This approach is very similar to the gradient solution method reported by Abubakar and Sagir (2013) and was chosen over similar methods due to its high probability of convergence to solutions over fewer iterations.

In the application of the EPANET model, the analysis is done by selecting initial flow estimates for every pipe in the system.

For every iteration in this method, nodal heads are determined by solving a matrix equation given as:

$$AH = F \tag{2.7}$$

Where:

A in the matrix equation = an (N x N) Jacobian matrix,

H = an (Nx1) vector of unknown nodal heads,

F = an (N x 1) vector of right-hand side terms.

After new heads are determined using the aforementioned matrix equation (2.7), the new flows through the pipes are determined. The advantage of this method is that it solves for the hydraulic parameters at every node within the system simultaneously. The criterion for convergence is user defined and is put in terms of a tolerance value. For example, if the sum of the absolute flow changes relative to the total flow through all of the pipes in the system is less than some prescribed tolerance then the process of solving the matrix equation and determining the new flows is terminated.

For instance, if a pipe network with *N* junction nodes and *NF* fixed grade nodes (tanks and Tanks) is considered, then the flow-head loss relation in a pipe between nodes *i* and *j* may be given as:

$$H_i - H_j = h_{ij} = rQ^n{}_{ij} \tag{2.8}$$

Where:

H = nodal head,

h = headloss,

r = resistance coefficient,

Q = flow rate,

n = flow exponent.

Three types of analyses which may be conducted using EPANET are: steady state (static), extended period (dynamic), and water quality analyses. The steady state analysis is used to compute the pipe flow rate and head loss in a steady state pipe network system. The extended period analysis simulates the continuous flow rate and pressure changes over a period of time. The water quality analysis is used to compute the age of water, perform source tracking, calculate the fate of a dissolved substance, or determine the growth or decay of a substance.

The calculation of flow rate and pressures for a steady state pipe network system is called a steady state analysis. This analysis computes the pipe flow rate and the hydraulic head loss so that the conservation of energy and mass are satisfied. In an extended simulation period, storage tanks and hydraulic switches are often present as part of the water distribution system. The system operating parameters at each time step depend on external conditions and the pipe flow rate from the previous time step. External conditions are operating parameters controlled by factors outside the system, such as external demand or pump power. The previous time step flow rates are also used to predict the storage tank levels for the current time step.

2.4.7 EPANET Water Quality Simulation Approach

The EPANET's water quality simulator uses a Lagrangian time-based approach to track the fate of discrete parcels of water as they move along pipes and mix together at junctions between fixed-length time steps. These water quality time steps are typically much shorter than the hydraulic time step (e.g., minutes rather than hours) to accommodate the short times of travel that can occur within pipes. The method tracks the concentration and size of a series of non-overlapping segments of water that fills each link of the network. As time progresses, the size of the most upstream segment in a link increases as water enters the link while an equal loss in size of the most downstream segment occurs as water leaves the link.

For each water quality time step, the contents of each segment are subjected to reaction, a cumulative account is kept of the total mass and flow volume entering each node, and the positions of the segments are updated. New node concentrations are then calculated which include the contributions from any external sources. Finally, a new segment is created at the end of each link that receives inflow from a node if the new node quality differs by a user-specified tolerance from that of the link's last segment. Initially each pipe in the network consists of a single segment whose quality equals the initial quality assigned to the upstream node.

Whenever there is a flow reversal in a pipe, the pipe's parcels are re-ordered from front to back. The simulation of chlorine decay in a distribution system requires the summed effects of the

bulk liquid and pipe wall. Even though zero, first, and second-order decay reactions are used in practice, a first-order reaction is widely accepted during simulation of chlorine decay. The first-order decay exponential equation (Ababu *et al*., 2019; Walski *et al*., 2003) is given as:

$$C_t = C_0^{-kt} \tag{2.9}$$

Where:

C_t = Concentration at time t (mg/l),

C_0 = Initial Concentration (at time zero) (mg/l),

kt = Reaction rate constant.

The reaction rate constant (k) is the overall reaction rate constant, in that it incorporates both the bulk K_b and wall K_w reaction rate constants.

Of the four programs, only EPANET and DEW can perform system flow dynamic simulation over an extended period of time. In addition to hydraulic simulation, EPANET can perform water quality analysis. EPANET has become a popular tool in analyzing complex and simple water distribution networks in both the developed and developing countries of the world (Adeniran & Oyelowo, 2013).

2.5 Model Calibration

Calibration can be defined as the process of comparing a model's results to field observations. If necessary, the input parameters describing a system can be adjusted until the model makes predictions that agree reasonably with measured values based on chosen statistical criteria. The parameters that may need adjustment may include but are not limited to: system demands, pipe roughness, and pump operating characteristics. Differences between a model and field observations could stem from poor field data that need to be corrected (Waski, 2017).

2.5.1 Hydraulic Calibration

Hydraulic simulation models are widely used for analysing the behaviour of water distribution systems. Due to the high degree of uncertainty and lack of details of the system, reliable management may be achieved only with an accurate calibrated model (Zanfei *et al.,* 2020). However, a modeler cannot make the assumption that the model is performing accurate simulations. Thus, the quality of the simulations is dependent on the quality of the data. Therefore, the accuracy of simulation software depends on how well it was calibrated. Performing a detailed calibration ensures that the model generates results that are accurate and reliable as per the selected statistical criteria. However, if the data are considered valid, the model inputs must be modified to minimize or at least perform as the set criteria.

2.5.2 Water Quality Calibration

Ensuring that a model is calibrated with respect to water quality simulation is a major concern to any modeler. A correctly calibrated water quality model is essential for accurate and reliable results. In order to calibrate a water quality model for a reactive constituent, the governing parameters for reaction must be correctly adjusted for the simulated results to closely match the measured results. However, Nejjari *et al.* (2014) reported that the calibration of water quality simulation should not be done until the hydraulic simulation component of the model has been completely calibrated. This is because water quality simulation is carried out for an extended period analysis, which requires water demands to vary over time, and this is achieved during hydraulic simulation.

One of the major challenges issue in municipal water quality modeling is gathering the essential input parameters of the model, particularly bulk decay (Kb) and wall decay (Kw) coefficients as well as their calibrations (Roya *et al.*, 2019). For the bulk reaction rate, the parameters of interest were the bulk reaction coefficient, bulk reaction order, and concentration limit. These parameters need to be adjusted for both the pipe and tank components in a system.

To simplify the calibration process, pipes with similar characteristics (i.e. pipe material and age) were combined into one calibration group for the bulk reaction coefficient adjustment. However, the tank bulk reaction coefficient was calibrated individually for each storage tank. The parameters that need to be adjusted for pipe wall reaction were the wall reaction coefficient and reaction order. Both of these parameters are related to pipe material and pipe wall conditions. This study used two different means of calibrating the pipe wall parameters.

The first method was known as direct calibration, which is similar to the calibration of the bulk reaction parameters and involved the grouping of pipes with similar characteristics (age, material, and location) and directly optimizing the pipe wall reaction coefficient and reaction order. The second method was known as correlation calibration and involved the assumption that there is a relationship between the increase of pipe wall roughness due to age and the reactivity of the pipe wall. The relationship between the pipe wall reaction coefficient and the pipe roughness varies depending on which head loss equation is used (Mays, 2004). These relationships are given by:

Hazen-Williams: $$\mathrm{Kw} = \frac{\mathrm{F}}{\mathrm{C}} \tag{2.10}$$

Darcy-Weisbach: $$\mathrm{Kw} = -\frac{\mathrm{F}}{\log\left(\frac{\mathrm{e}}{\mathrm{d}}\right)} \tag{2.11}$$

Chezy-Manning: $$\mathrm{Kw} = \mathrm{FN} \tag{2.12}$$

Where:

K_w = Pipe wall reaction coefficient,

C = Hazen-Williams C-factor,

e = Darcy-Weisbach roughness,

N = Manning roughness coefficient,

d = Pipe diameter (m),

F = Coefficient of correlation.

The coefficient of correlation, F, is related to the wall reaction coefficient in a way that is dependent on the head loss equation used. The parameter F must be determined from site-specific field measurements. The advantage of using correlation calibration is that it requires only a single parameter F, to allow wall reaction coefficients to vary throughout a system in a physically meaningful way.

Based on the above Literature Review the actual pipe roughness factor within water distribution networks has rarely been periodically determined as one of the factors to establish and predict the hydraulic performance of the system. In this regard the research gap of this work is the periodic determination of the pipe roughness factors for each pipe in a water distribution network in order to ensure water availability to communities as per the United Nations Sustainable Development Goal 6.

CHAPTER THREE
METHODOLOGY

3.1 Study Area

This study was carried out for the Maralal water distribution system located in Maralal town. Maralal town is the administrative and commercial centre of Samburu County, one of the 47 Counties in Kenya. It is located at longitudes 36041', 36043'and latitudes 01005',01007' at an elevation of 1940m as shown in Figure 3.1.

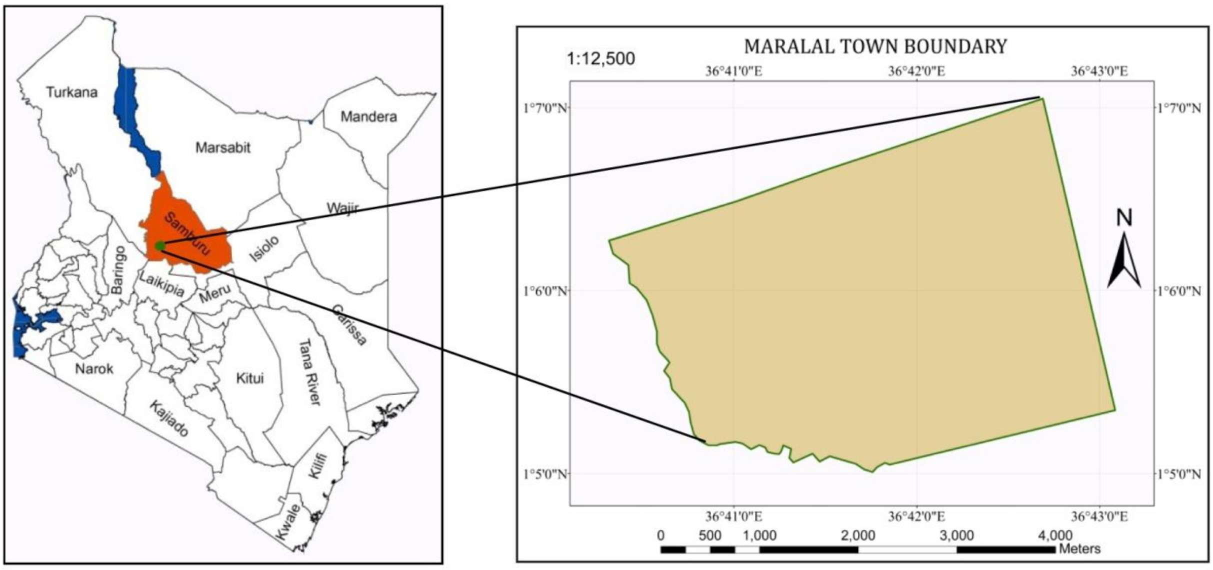


Figure 3.1: Map of the Study Area (Source: Geocurrents)

The town has great potential for further development considering its commercial and trading opportunities. This is in due to its location as well as being the biggest town in Samburu County. The only other nearest big town is Nyahururu that is located about 160km away. Maralal town is located on a relatively sloping terrain ranging in elevation from 1900 to 2200m above mean sea level (a.m.s.l.). It receives a mean annual rainfall of about 605mm while its maximum temperature is 33.4^0C. Maralal town is underlain by rock formation that is mainly the tertiary Volcanics of the Rift Valley.

The town has experienced an increase in population over the last years. The population of Maralal town by 2019 was 31,350 people (KNBS, 2019). The population growth rate is approximated to increase at 7.5% per annum (KNBS, 2019). Due to the rapid increase of population in Maralal town over the years, the existing water supply is inadequate in meeting the town's water demand. The current water coverage is estimated at about 22% as shown in Table 3.1.

Table 3.1: Water Demand and Supply within Maralal Location

Name of Sub location	**Water Demand (m^3/day)**	**Water supply (m^3/day)**	**Demand Coverage (%)**
Lpartuk	428	114	26.6
Lkuroto	317	70	22.0
Ledero	324	0	0.0
Ngari	623	117	18.8
Maralal Town	1,541	410	26.6
Shabaa	762	158	20.7
Milimani	554	132	23.8
Total	4,556	1002	22.0

Source: MAWASCO Annual Report (2012)

The existing water distribution network comprises un-plasticized polyvinyl chloride (upvc) pipes of 200mm, 150mm and 100mm diameter, which was laid down in 1986. It is concentrated in an area of 6km^2 against the total town area of about 150km^2 as shown in Figure 3.2.

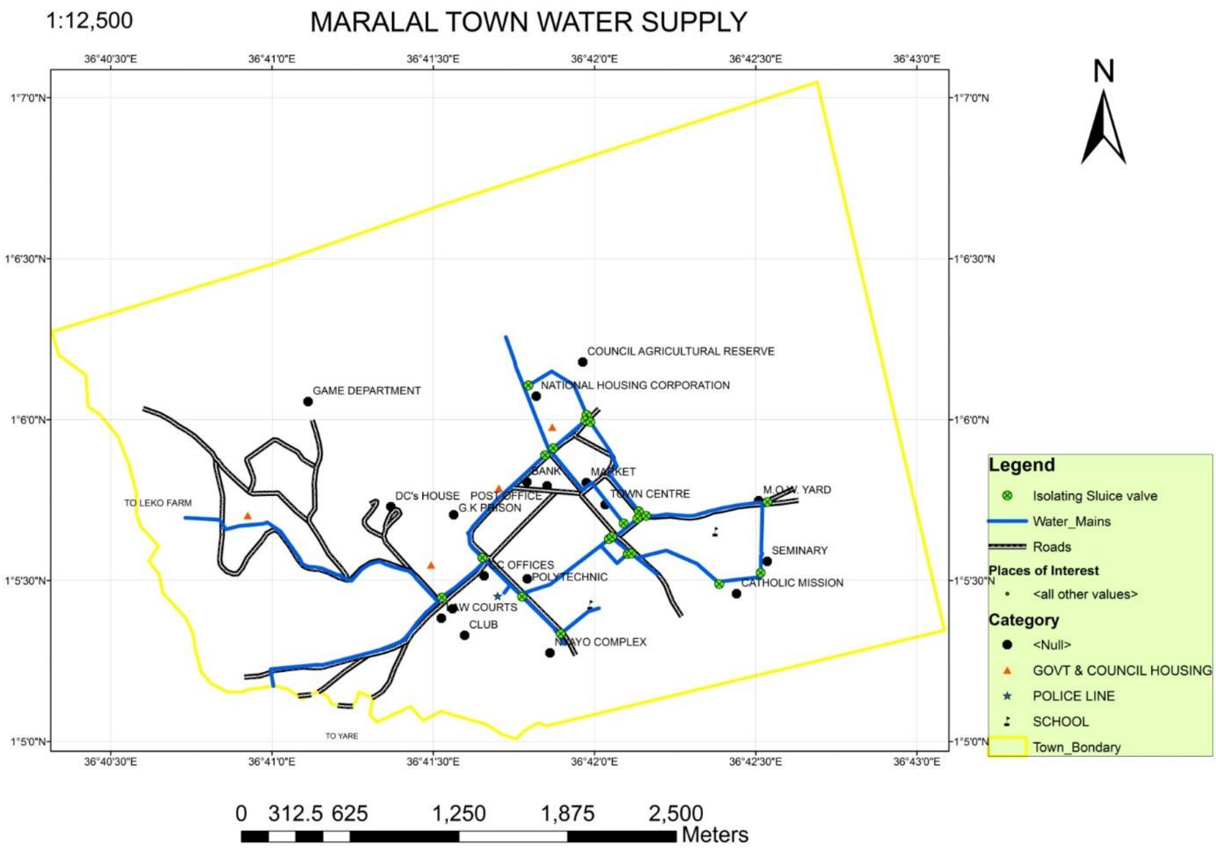


Figure 3.2: Maralal Water Distribution Network

3.2 Determination of roughness factors of each pipe of the network

3.2.1 Data Collection for Maralal Water Distribution System

The data collected for this study included elevations, pipe diameters, pipe material, pipe lengths, flow rate, pressure, head loss, and initial and residual chlorine concentrations. The data was collected at different sub-sections of the network. For the purposes of this study the data was collected along pipes, at junctions, at Tanks and at control valves.

Some of this data that was used in the model was collected from the field while the rest was secondary data sourced from existing network information. For instance, the elevations for the study site were established from the field through surveying of the water distribution system. To obtain this, a handheld Garmin E-trex Vista Global Positioning System (GPS) device was used. The pipe lengths and diameters were determined from the network design drawings from the Maralal Water and Sanitation Company (MAWASCO) offices.

The principal hydraulic data for the pipes that was collected included: start and end nodes, pipe diameters and lengths. In addition, the flow rates and pressure were measured. The collected data was used to determine the roughness coefficient for each pipe within the water distribution system. In addition to collection of data concerning the hydraulic pipe parameters, the water quality data was also collected for pipes. The water quality data that was measured consisted of initial and residual chlorine concentrations. The data on the junctions included their elevation, amount of water through each junction and the quality of water. Data on tanks which acted as nodes within the network involved their diameters, shape, initial, minimum and maximum water levels, initial and final chlorine concentrations.

During the fieldwork, the Maralal distribution network was divided into three (3) zones as per the water distribution hedging programme developed by the WSP, namely; County, PCEA, Towns 1 and 2.

The flow rate was measured along 9 pipes after every 20minutes for four hours per day by use of a water meter (Kent PR7P-1) where the pressure was measured in terms of meter-head of water using a 10bar water pressure gauge (see Figures 3.3 and 3.4). Pressure was measured at eight (8) nodes within the Maralal water distribution network at an interval of 20 minutes for four (4) hours for six (6) days. Each pipe had a start and an end node.

The difference in pressure between any two successive points and the elevation difference were used to compute the head loss. In addition, initial and residual chlorine concentrations were measured at PIPE9 using a lovibond comparator (Tintometer 2000+). Calibrated comparator discs were used to match the sample concentration with a control so that the matching value was the sample chlorine concentration in mg/l. The values were used as the EPANET model inputs for the simulation of the water distribution network.

Figure 3.3: Installation of a Water Meter for Flow Measurement

Figure 3.4: Water Meter and Pressure Gauge for Flow and Pressure Measurement respectively

Table 3.2 shows the summary parameters that were measured and equipment that were used.

Table 3.2: Parameters and Measurement Tools

Measurements	Materials/Tools
Elevation (m)	GPS, Maps
Pipe diameters, pipe lengths and pipe material (mm)	Reticulation Maps
Flow rate (m^3/s)	Water meter
Pressure (m)	Pressure gauge
Headloss (m)	Computed
Initial and Residual chlorine(mg/l)	Lovibond comparator

3.2.2 Regression Analysis of Calculated Headloss and Measured Flow Rate

Using equation (2.4) section 2.2.2, the parameters A and B were determined using the regression analysis of the measured head loss and flow rate. This led to the relationship between head loss and flow rate determined. The current roughness factor C was determined using the equation given as:

$$A = C_f C^{\alpha} d^{\rho} L \tag{3.1}$$

Where:

A = Resistance coefficient,

C_f = Units conversion factor (10.29 for SI units),
C = Determined Roughness factor,

d = pipe diameter in meter (m),

α = Roughness exponent,
ρ = diameter exponent (Hazen Williams equation α=-1.852 and ρ=-4.871),
L = Pipe length (m).

The determined roughness factors (C) were then used in the model for simulation and prediction of the system performance.

3.3 Simulation and Prediction of Steady and Dynamic State Hydraulic Parameters

Water distribution system simulation process was conducted by first identifying the service needs for which solutions were determined.

Model demands were set up manually as the model required inputs that are discussed under section 3.3.2 below. After model calibration and validation, the model was run for different scenarios and the output for each scenario was obtained. The management processes that were used for the simulation are illustrated in Figures 3.5 and 3.6 respectively.

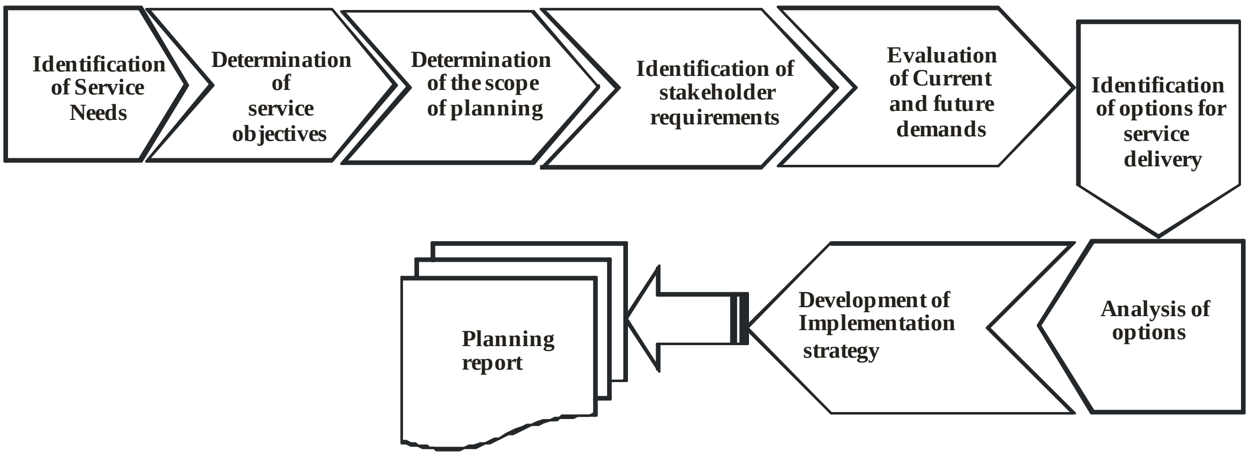


Figure 3.5: The Simulation Process

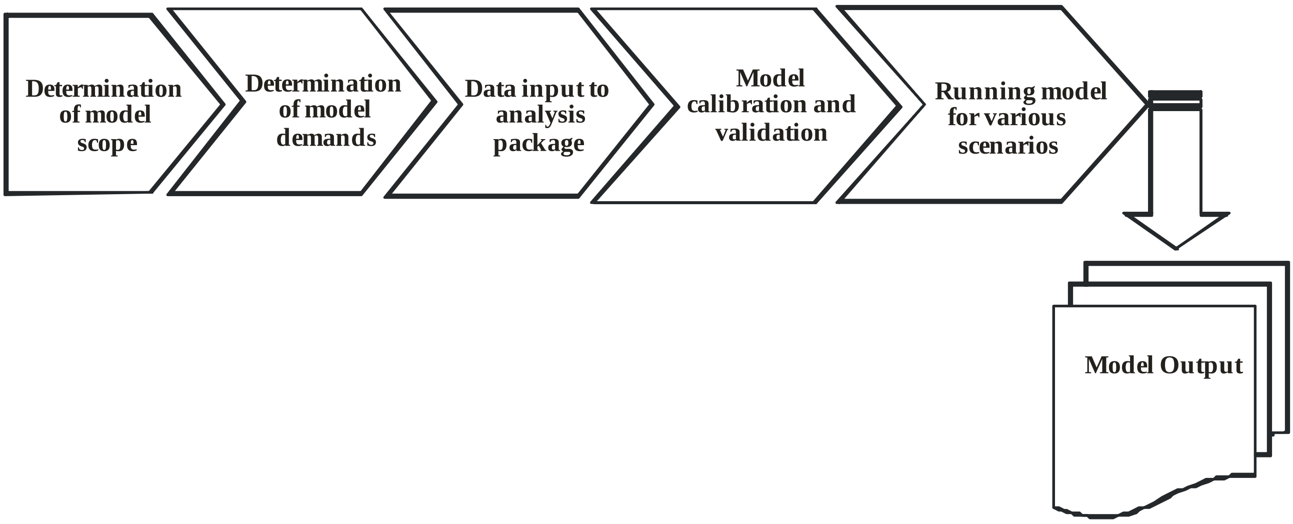


Figure 3.6: Model Management Process

3.3.1 Water Flow Distribution Simulation Methods

Three types of analyses namely; the steady state (static), extended period (dynamic) and water quality analyses were conducted. The steady state analysis was used to compute the flow rate in the pipe, head loss and the node pressure head in a steady state pipe network system in which conservation of energy and mass are satisfied. The extended period analysis simulated the continuous flow rate and pressure changes in the distribution system over a period of time. The total simulation time was divided into two-time steps. At each time step, an analysis was conducted for the pipe network based on parameters of the current network and the pipe flow rate calculated from the previous time step. The water quality analysis was done to simulate the decay in chlorine concentrations thus determining residual levels.

Data collected from field measurements was entered into EPANET computer model. The EPANET computer model which is composed of two parts: the input data file and; the computer program was used for analyzing the water distribution. Coefficients of roughness as Hazen-William C factor determined in section 3.2.2 were used as an input to the model.

The first task was to create a new project in EPANET environment which was initiated by selecting the default options in the model menu. A schematic drawing of the network as shown in Figure 3.7 was done using the relevant drawing tools available in the model. The schematic drawing represented the Maralal water distribution system in the virtual EPANET model environment. This network was a collection of links connected to nodes with links representing pipes while the nodes represented junctions and tanks. As objects were added to the project they were assigned a default set of properties during the initial simulation. To change the value of a specific property for an object, the object was selected into the *Property Editor* in order to insert the required property. Figure 3.7 shows the Maralal virtual water distribution network in EPANET model environment. Figure 3.7 indicates County zone was served by PIPE0, PIPE1 and PIPE2 while PCEA zone was served by PIPE0, PIPE7, PIPE8 and PIPE9. TOWN 1 zone was served by PIPE0, PIPE1, PIPE3 and PIPE4 with TOWN 2 zone served by PIPE0, PIPE1, PIPE6 and PIPE5.

Each Zone received water services independently with all sluice valves leading to other zones being closed

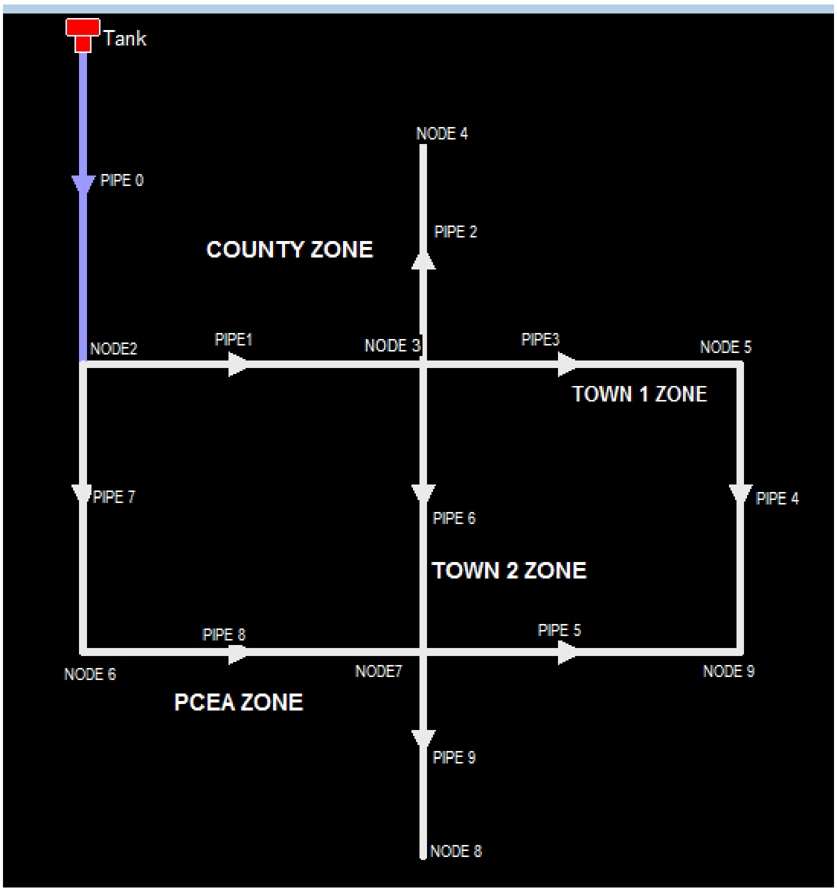


Figure 3.7: Components of Maralal Water Distribution System using the EPANET Model

The basic input data for junctions was: the site elevations; water demand (rate of withdrawal from the network) and initial water quality. The primary input properties for the tank were the hydraulic head (equal to the water surface elevation if the Tank is not under pressure) and the initial quality. The principal hydraulic input parameters for the pipes were: the start and end nodes for each pipe; diameter; length; roughness coefficient (for determining head loss) and the status of the pipe (open, closed, or contains a check valve).

The junction heads, head losses and pipe flows for a fixed set of Tank levels and water demands over a succession of points in time were simulated using the EPANET's hydraulic simulation program.

To make the network more realistic for analyzing an extended period of operation, a Time Pattern was created. A pattern of selected time step was used thus making demands change at different times of the day.

The results of the model in regard to flow rate and headloss were compared with the measured data from the field.

3.3.2 Model Calibration

Calibration was performed with the purpose of comparing the simulated data from the model with field data. Both the hydraulic and water quality calibrations were performed but the calibration of water quality simulation was done after the hydraulic simulation component of the model had been completed. The head loss and flow rate were the hydraulic parameters that were used for calibration, while residual chlorine concentration was the water quality parameter which was used during calibration.

To achieve the calibration, nodal demands and wall reaction coefficients were adjusted until the model output and the results measured data closely matched according to the Nash-Sutcliff model efficiency.

The Nash-Sutcliffe model efficiency coefficient (E) was used to assess the predictive ability of the model. The efficiency coefficient equation is given as:

$$E = 1 - \frac{\sum_{i=1}^{n}(X_{obs,i} - X_{model})^2}{\sum_{i=1}^{n}(X_{obs,i} - \overline{X_{obs}})^2} \tag{3.2}$$

Where:

X_{obs} = observed values and

X_{model} = modeled values at time/place i.

Since Nash-Sutcliffe efficiencies range from -∞ to 1, an efficiency of 1 (E = 1) was used to correspond to a perfect match between model and observations.

3.3.3 Model Validation

The model was validated by verifying the output results from measured parameters using different sets of measured data. The model was run using different scenarios for the distribution network to predict the impact of any changes on the network to the hydraulic parameters of the system. To achieve this, two sets of data were used to validate the model.

3.3.4 Model Sensitivity

Independent variables were varied in order to note the changes in the output dependent variables. The independent variables that were varied were; pipe roughness, pipe diameter, pipe length and initial chlorine concentrations. Each of these variables was reduced and also increased by 10% for the entire network, and the changes to the output variables were presented graphically in Chapter Four.

3.4 Impact of Chlorine Decay on System Hydraulic Performance

Transport and decay of chlorine through the network were simulated using EPANET's water quality simulator**.** However, in order to carry out the simulations, appropriate data was entered in the model data base. The reaction rate constant (k) was the overall reaction rate constant, in that it incorporated both the bulk K_b and wall K_w reaction rate constants.

The K_b for first-order reactions was determined through the laboratory tests by placing a sample of water in a series of non-reacting glass bottles and analyzing the contents of each bottle at different times.

Figure 3.8: Residual Chlorine Measurement at Maralal Water Quality Laboratory

Since the reaction was first-order, then plotting the natural log (C_t/C_o) against time resulted in a straight line, where C_t was concentration at time t and C_o was concentration at time zero. K_b was the slope of the line. The wall reaction coefficients K_w and initial chlorine concentration C_o were set to selected values.

The simulation was performed and the time controls on the map browser were used to show how chlorine levels were changing spatially and temporally throughout the simulation period.

Distributions of chlorine levels were analyzed with respect to pipe diameter and its impact on hydraulic parameters of the distribution was determined.

The chlorine levels were compared with respective section head loss to establish the impact of chlorine decay on the flow dynamics determined.

CHAPTER FOUR
RESULTS AND DISCUSSION

4.1 Roughness Factors of the Pipe Network

Roughness factors for each pipe were determined using regression analysis of the calculated headloss and measured flow rate results. For instance, the regression equation for PIPE 1 in County zone was $h_L=13599.68q^{1.85}$. Headloss was calculated from measured pressure.

4.1.1 Measured Flow and Pressure Results

Network pipe and Node properties were collected and recorded as shown in Tables 4.1 and 4.2 respectively. Flow and Pressure results were recorded as shown in Tables 4.3, 4.4 and 4.5 below.

Table 4.1: Network Pipe Properties

PIPE ID	LENGTH (m)	DIAMETER (mm)
PIPE0	2200	200
PIPE1	874	150
PIPE2	396	100
PIPE3	355	100
PIPE4	755	100
PIPE5	220	100
PIPE6	676	100
PIPE7	356	100
PIPE8	510	100
PIPE9	754	100

Table 4.2: Node Properties

NODE NOTATION	ELEVATION (m)
Tank	2040
NODE2	1956
NODE3	1953
NODE4	1980
NODE5	1955
NODE6	1951
NODE7	1947
NODE8	1945
NODE9	1945

Table 4.3: Flow Rates (m^3/hr) County Zone Pipe 1

DAY 1	DAY 2	DAY 3	DAY 4	DAY 5	DAY 6	AVERAGE
55.85	54.40	54.91	56.93	58.28	57.61	56.33
60.16	59.70	58.75	61.04	61.13	60.62	60.23
65.22	64.35	64.09	64.31	64.79	65.05	64.64
65.44	64.57	64.13	64.57	64.96	65.35	64.84
65.57	65.14	64.48	65.14	65.57	65.91	65.30
66.77	66.13	65.91	66.98	66.60	66.81	66.53
67.78	66.94	66.09	66.60	67.36	67.44	67.03
69.03	68.20	67.86	68.66	68.61	68.82	68.53
69.32	68.90	68.28	68.78	68.90	69.15	68.89
71.19	70.38	69.97	70.42	71.11	70.95	70.67
72.46	72.27	71.79	72.27	72.62	72.70	72.35
72.78	72.38	72.46	72.35	72.86	73.33	72.69
Av.66.80	**66.11**	**65.73**	**66.50**	**66.90**	**66.98**	**66.50**

Table 4.3 shows flow results from measurements taken at an interval of 20minutes for four hours each day for six days for County zone along PIPE1. The measurements were taken during the peak flow between 10am-2.00pm when the flow was steadily increasing based on the opening of the outflow sluice valve of the main tank. The sluice valve was not opened

fully at once but rather opened intermittently in order to avoid pipe bursts due to rapid pressure. The average flows along PIPE1 in County Zone for days 1, 2, 3, 4, 5 and 6 were 66.80, 66.11, 65.73, 66.50, 66.90 and 66.98m^3/hr respectively. The average flow along Pipe1 for the six days was 66.50m^3/hr. The rest of the flow results can be found in Tables A8, A9, A10 and A11 in Appendix A.

Table 4.4: Pressure Results (m) for Nodes 2 and 3 along PIPE1 in County Zone

DAY1		DAY 2		DAY 3		DAY4		DAY5		DAY6	
N2	N3	N2	N3	N2	N3	N2	N3	N2	N3	N2	N3
58.55	49.45	58.46	49.65	58.36	49.06	58.86	49.54	59.16	49.76	58.96	49.50
58.44	48.44	58.44	48.54	58.34	48.64	58.74	48.55	58.75	48.54	58.65	48.55
58.23	47.10	58.13	47.20	58.03	47.16	58.13	47.21	58.23	47.20	58.33	47.24
57.18	46.00	57.08	46.10	57.00	46.12	57.10	46.12	57.18	46.11	57.28	46.12
56.21	45.00	56.21	45.10	56.11	45.15	56.23	45.12	56.31	45.10	56.38	45.09
56.05	44.56	56.00	44.66	55.95	44.66	56.10	44.56	56.11	44.66	56.18	44.78
55.85	44.12	55.75	44.22	55.65	44.32	55.77	44.32	55.85	44.22	55.85	44.20
55.18	43.15	55.08	43.25	55.00	43.25	55.18	43.24	55.18	43.25	55.28	43.30
54.10	41.00	54.10	42.10	53.95	42.10	54.12	42.15	54.10	42.10	54.18	42.12
53.56	40.00	53.46	41.10	53.36	41.10	53.47	41.10	53.56	41.52	53.60	41.41
52.88	39.00	52.78	39.95	52.68	40.00	52.79	40.23	52.88	40.56	52.90	40.53
51.96	38.00	51.86	39.00	51.76	39.00	51.93	39.26	51.96	39.58	52.20	39.76

From Table 4.4, the results show that there was a pressure drop both over time in each node and between successive nodes in each day. The pressure drops over time in each node was due to increased flow rate that was occasioned by the gradual opening of the tank sluice valve which in turn increased headloss due to friction along the respective pipes. The cumulative headloss along preceding pipes and elevation difference caused the drop in pressure from NODE2 (N2) to NODE3 (N3) in each day. This can be illustrated by considering the results of Day 1 above for both N2 and N3. The N2 pressure results indicate that pressure dropped in every 20 minutes from 58.55m to 51.96m while N3 pressure dropped from 49.45m to 38m. More so, the pressure dropped from 58.55m at N2 to 49.45m at N3. This similar attribute can also be seen vertically and horizontally over time in a node and between nodes respectively in the rest of the days. The rest of the Pressure results can be found in Tables A1, A2, A4 and A6 in Appendix A.

4.1.2 Results of Head Loss

Head loss along any pipe (n) was calculated using equation 4.1, where the sum of pressure drop and elevation difference between any two successive nodes was taken as the head loss between the two nodes.

$$HLn = (ET - Ej) - Pj - HL(n-1) - HL(n-2) - \cdots \ldots..HL1 \tag{4.1}$$

Where; H_{Ln} = Headloss along pipe n

E_T = Tank Elevation

i= Start node of pipe n

j= End node of pipe n

E_j = End node elevation

P_j = Pressure at End node

H_{L1} = Headloss along start pipe (pipe 0).

Table 4.5: Headloss (m) County Zone Pipe 1

Day 1	Day 2	Day 3	Day 4	Day 5	Day 6	Average
6.10	5.81	5.91	6.32	6.60	6.46	6.20
7.00	6.90	6.70	7.19	7.21	7.10	7.02
8.13	7.93	7.87	7.92	8.03	8.09	8.00
8.18	7.98	7.88	7.98	8.07	8.16	8.04
8.21	8.11	7.96	8.11	8.21	8.29	8.15
8.49	8.34	8.29	8.54	8.45	8.50	8.44
8.73	8.53	8.33	8.45	8.63	8.65	8.55
9.03	8.83	8.75	8.94	8.93	8.98	8.91
9.10	9.00	8.85	8.97	9.00	9.06	9.00
9.56	9.36	9.26	9.37	9.54	9.50	9.43
9.88	9.83	9.71	9.83	9.92	9.94	9.85
9.96	9.86	9.88	9.85	9.98	10.10	9.94

Table 4.5 presents values obtained as headloss for County zone along PIPE1. The results show that the headloss increased over time for the six days. This is the case due to the exponential relationship between headloss and flowrate with the later increased by the gradual opening of the Tank sluice valve. Headloss values for Day 1 increased from 6.10m to 9.96m while in Day 2 it increased from 5.81m to 9.86m. Headloss for Days 3 and 4 increased from 5.91 to 9.88m and from 6.32m to 9.85 respectively. On Day 5, the headloss increased from 6.60m to 9.98m while Day 6 increased from 6.46m to 10.10m during the 4-hour period.

The average headloss values for PIPE1 for the six days for the 20-minutes-interval 4-hour period were 6.20m, 7.02m, 8.00m, 8.04m, 8.15m, 8.44m, 8.55m, 8.91m, 9.00m, 9.43m, 9.85m and 9.94m. The same determination was made for all the nine pipes of the network and the results are found in Tables A1, A3, A5 and A7 in Appendix A.

4.1.3 Average Hydraulic Parameters for Pipes and Nodes

Figure 4.1 illustrates County zone water distribution network where water flows from the Tank through NODE2, NODE3 and NODE4 along PIPE0, PIPE1 and PIPE2. Hydraulic parameters values measured in pipes and nodes within county zone water distribution network are shown in Table 4.6.

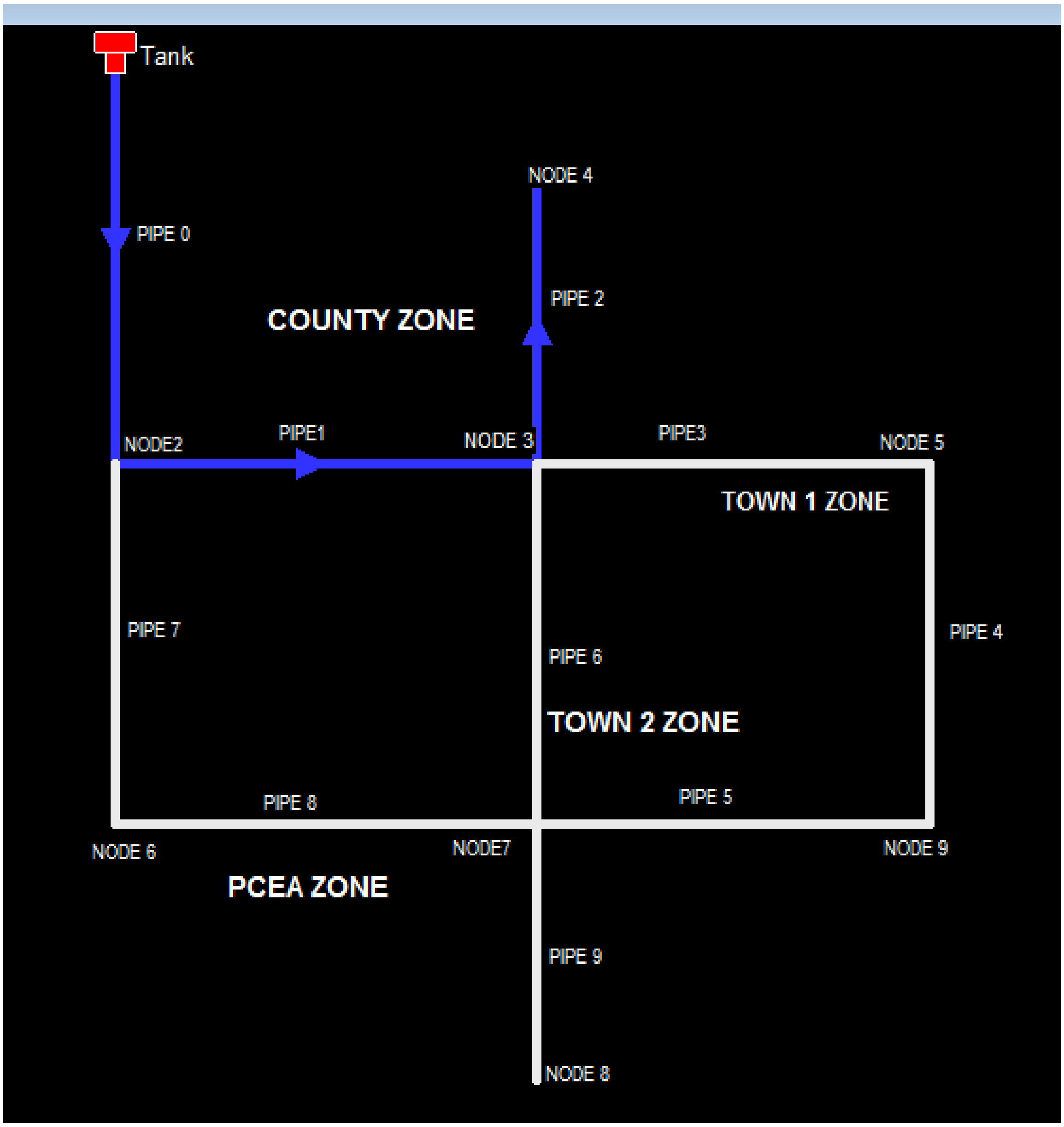


Figure 4.1: County Zone Pipes and Nodes

Table 4.6: County ZONE Average Hydraulic Parameters Results

FLOW RATE (m^3/hr)		PRESSURE (m)			HEADLOSS (m)	
P1	P2	N2	N3	N4	P1	P2
56.33	30.03	58.73	49.49	15.55	6.20	6.02
60.23	30.86	58.56	48.54	14.21	7.02	6.33
64.64	31.02	58.18	47.19	12.79	8.00	6.40
64.84	31.29	57.14	46.10	11.60	8.04	6.50
65.30	31.56	56.24	45.09	10.49	8.15	6.60
66.53	31.76	56.07	44.65	9.95	8.44	6.68
67.03	32.26	55.79	44.23	9.33	8.55	6.88
68.53	32.53	55.15	43.24	8.22	8.91	6.98
68.89	32.74	54.09	41.93	6.83	9.00	7.06
70.67	32.91	53.50	41.04	5.80	9.43	7.13
72.35	33.04	52.82	40.05	4.66	9.85	7.19
72.69	33.21	51.95	39.10	3.59	9.94	7.26

From Table 4.6 the results show that the flow rate increased from 56.33 m^3/hr to 72.69 m^3/hr and from 30.03 m^3/hr to 33.21 m^3/hr for PIPE1 and PIPE2 respectively. Similarly, headloss increased from 6.20m to 9.94m and from 6.02m to 7.26m for PIPE1 and PIPE2 respectively. However, pressure depicted a decreasing trend from 58.73m to 51.95m, 49.49m to 39.10m and 15.55m to 3.59m for NODE2, NODE3 and NODE4 respectively.

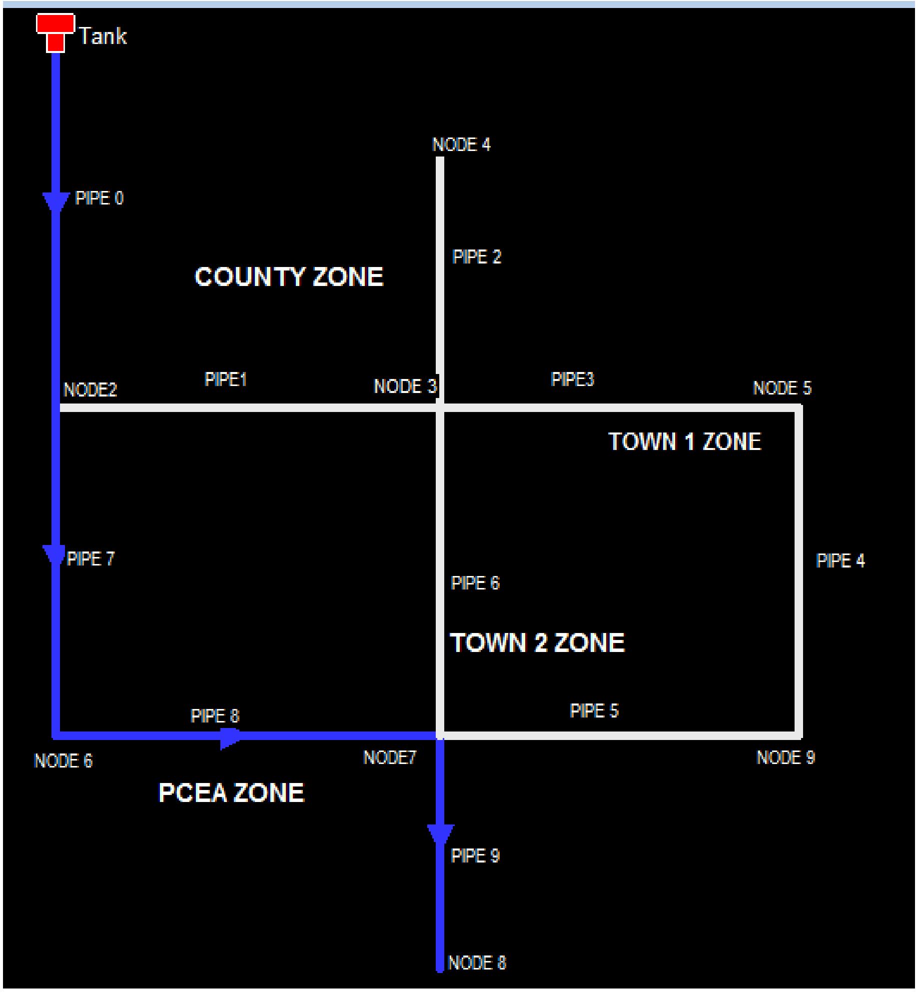


Figure 4.2: PCEA Zone Pipes and Nodes

Figure 4.2 indicates PCEA zone water distribution network where water flows from the Tank through NODE2, NODE6, NODE7 and NODE8. PIPE0, PIPE7, PIPE8 and PIPE9 are the water distribution pipes. Hydraulic parameters values measured in pipes and nodes within PCEA zone water distribution network are shown in Table 4.7.

The flow rate results depict a decreasing trend across succeeding pipes but an increasing trend along each pipe. Flow for the first 20-minutes interval across PIPE7, PIPE8 and PIPE9 are 51.55m, 51.28 and 14.77m respectively. However, flow values for PIPE7 ranged from 51.75m to 55.69m while PIPE8 and PIPE9 ranged from 51.28 to 55.46m and from 14.77m to 19.68m respectively.

Table 4.7: PCEA ZONE Average Hydraulic Parameters Results

FLOW RATE (m^3/hr)			PRESSURE (m)				HEADLOSS (m)		
PIPE7	PIPE8	PIPE9	NODE2	NODE6	NODE7	NODE8	PIPE7	PIPE8	PIPE9
51.75	51.28	14.77	57.88	48.05	30.50	29.42	15.04	21.80	3.08
52.07	51.43	15.23	57.07	47.07	29.43	28.17	15.15	22.01	3.19
52.20	51.60	15.46	55.85	45.78	28.07	26.71	15.18	22.05	3.26
52.60	51.80	16.06	55.07	44.79	26.92	25.32	15.32	22.20	3.62
52.79	51.89	16.91	54.43	44.04	26.02	24.06	15.44	22.34	3.78
52.96	52.36	17.14	53.10	42.62	24.43	22.37	15.54	22.47	4.06
54.20	53.71	17.41	51.09	39.93	21.23	19.05	16.66	22.73	4.14
54.37	53.92	17.86	50.08	38.83	20.31	17.93	16.70	22.87	4.94
54.78	53.95	18.47	48.57	37.08	18.00	15.34	16.82	23.14	4.99
55.10	54.67	18.84	47.40	35.74	16.15	13.32	16.90	23.82	5.15
55.48	55.39	19.10	45.98	34.11	14.69	11.73	17.22	24.08	5.23
55.69	55.46	19.68	44.72	32.73	13.20	10.04	17.34	24.28	5.41

Pressure results show a decreasing trend both through and across the nodes. Pressure dropped from 57.88m to 44.72m at NODE2 and at NODE6, NODE7 and NODE8 there was a drop from 48.05m to 42.73m, from 30.50m to 13.20m and from 29.42 to 10.04 respectively. Similarly, a pressure drop was noted across the succeeding nodes where NODE2, NODE6, NODE7 and NODE8 values were 57.88m, 48.05m, 30.50m and 29.42m respectively.

However, headloss results show a similar increasing trend as the flow rate values along pipes with a varied change across the pipes. Headloss values increased along PIPE7, PIPE8 and PIPE9 from 15.04m to 17.34m, from 21.80m to 24.28m and from 3.08m to 5.41m respectively but values varied across the pipes.

Hydraulic parameters values measured in pipes and nodes within Town 1 zone water distribution network are shown in Table 4.8.

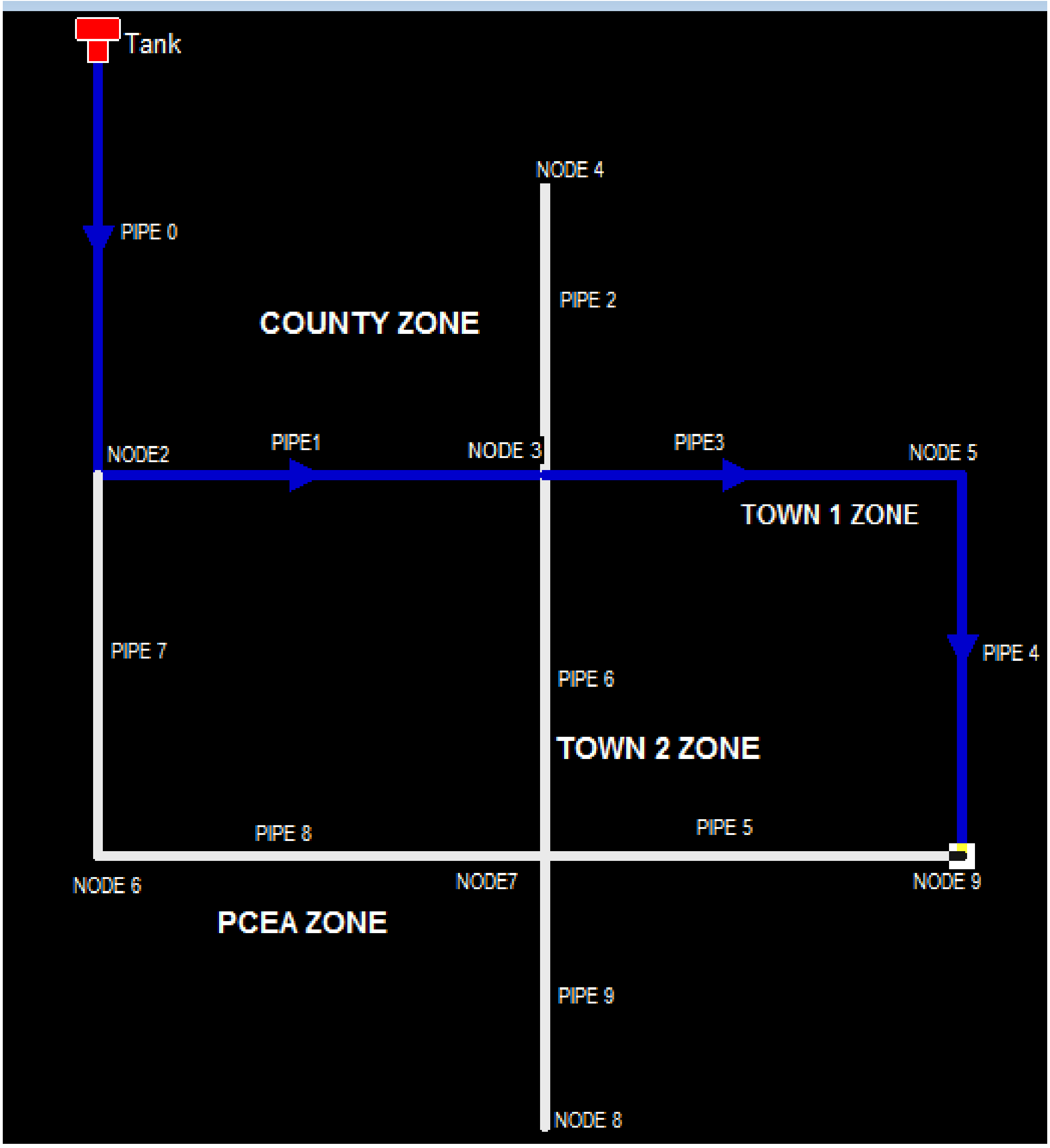


Figure 4.3: Pipes and Nodes for TOWN 1 Zone

Table 4.8: Town 1 Zone Average Hydraulic Results

FLOW RATE (m^3/hr)			PRESSURE (m)				HEADLOSS (m)		
P1	P3	P4	N2	N3	N5	N9	P1	P3	P4
82.25	64.52	24.69	56.62	48.77	23.82	26.05	12.95	24.00	7.99
82.94	64.80	24.86	56.05	48.06	22.92	24.93	13.26	24.55	8.09
83.72	65.12	24.91	57.34	47.43	22.08	24.01	13.47	24.65	8.12
85.63	65.63	25.42	57.14	46.68	20.98	22.84	14.22	25.24	8.43
86.76	65.77	26.43	56.66	45.86	20.08	21.82	14.33	25.40	9.06
86.88	66.00	26.94	56.39	45.56	19.62	20.93	14.37	25.68	9.39
87.12	66.70	27.08	56.11	45.21	18.79	20.03	14.49	26.24	9.48
87.44	66.69	27.25	55.57	44.58	18.17	19.11	14.57	26.35	9.59
87.79	66.92	27.57	55.14	44.04	17.47	18.10	14.68	26.54	9.80
88.18	67.21	28.28	54.84	43.62	16.85	17.21	14.79	26.67	10.27
88.89	67.50	28.34	54.75	43.32	16.36	16.68	14.82	27.01	10.31
89.50	67.86	28.44	54.32	42.71	15.50	15.65	15.06	27.23	10.38

From Table 4.8, it is observed that flow rate increases along each pipe but on the other hand it decreases along the zone network. PIPE1, PIPE3 and PIPE4 values increase from 82.25m^3/hr to 89.5m^3/hr, from 64.52m^3/hr to 67.86m^3/hr and from 24.69m^3/hr to 28.44m^3/hr respectively. However due to water demand flow during the first 20-minutes interval for PIPE1, PIPE3, and PIPE4 was 82.25m^3/hr, 64.52m^3/hr and 24.31m^3/hr respectively. However, pressure results decrease within each node but varies from node to node. Headloss increase along each pipe of the zone but vary along the network pipes. Headloss values for PIPE1, PIPE2 and PIPE3 increase from 12.95m to 15.06m, from 24.00m to 27.23m and from 7.99m to 10.38m respectively.

Increased flow rate was occasioned by the gradual opening of the tank sluice valve. Higher flow results caused an increase in headloss due to more friction along the respective pipes. The cumulative headloss along preceding pipes and elevation difference caused the decrease in pressure.

Figure 4.4 illustrates Town 2 zone water distribution network where water flows from the Tank through NODE3, NODE7 and NODE9 along PIPE0, PIPE1, PIPE6 and PIPE5. Hydraulic parameters values measured in pipes and nodes within Town 2 zone water distribution network are shown in Table 4.9.

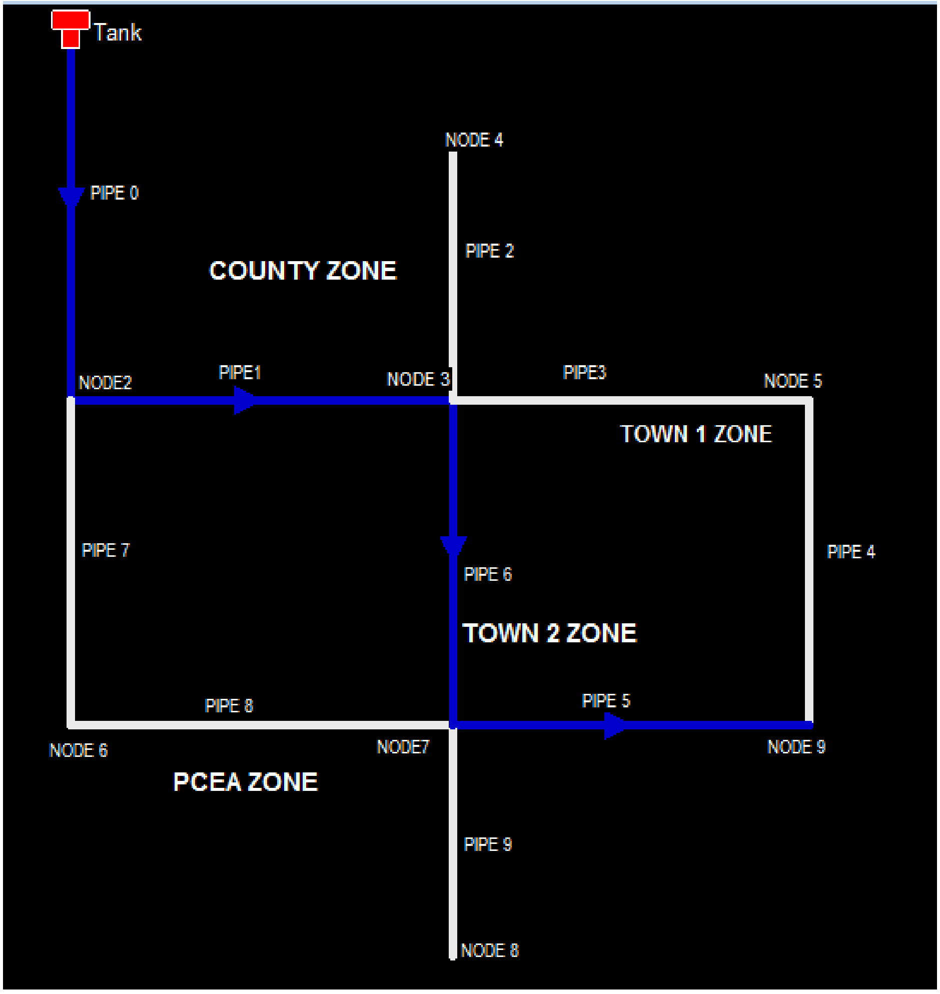


Figure 4.4: Pipes and Nodes for Town 2 Zone

Table 4.9 presents results of flow rate, pressure and headloss for Town 2 zone. From Table 4.9, the PIPE1 flow results range from 89.67m^3/hr to 95.48m^3/hr while for the PIPE6 and PIPE5 range from 58.20m^3/hr to 59.67m^3/hr and from 21.86 m^3/hr to 33.10m^3/hr respectively. For every 20-minutes interval of the 4-hour period flow values depict a decreasing trend. As presented in Table 4.9, the PIPE1, PIPE6 and PIPE5 flow rate values for the first 20-minutes interval are 89.67m^3/hr, 58.20m^3/hr and 21.86m^3/hr respectively. The pressure values present a decreasing trend in each node. Headloss values indicate an increasing trend for each pipe but show a varied trend from pipe to pipe.

Table 4.9: Town 2 Average Hydraulic Parameters Results

FLOW RATE (m^3/hr)			PRESSURE (m)				HEADLOSS (m)		
P1	P6	P5	N2	N3	N7	N9	P1	P6	P5
89.67	58.20	21.86	59.54	47.88	17.77	17.88	14.66	36.31	1.89
90.27	58.23	22.35	58.85	47.01	16.87	16.90	14.84	36.73	1.97
91.32	58.53	24.80	58.05	45.89	15.40	15.01	15.17	36.77	2.39
91.51	58.64	26.10	56.87	44.65	14.03	13.41	15.22	36.85	2.63
92.00	58.80	27.56	55.78	43.40	12.60	11.70	15.37	36.98	2.91
92.52	58.88	26.15	54.62	42.08	11.19	10.54	15.54	37.07	2.65
92.98	59.02	28.50	53.84	41.16	10.11	9.02	15.68	37.29	3.09
93.40	59.21	29.53	52.70	39.89	8.62	7.32	15.81	37.56	3.30
93.66	59.31	30.46	51.71	38.82	7.43	5.94	15.89	37.60	3.49
94.00	59.44	31.49	50.74	37.74	6.19	4.48	16.00	37.74	3.71
94.45	59.58	32.31	50.12	36.98	5.27	3.38	16.14	37.99	3.89
95.48	59.67	33.10	49.13	35.66	3.84	1.77	16.47	38.36	4.07

4.1.4 Regression of Flow and Headloss

A regression analysis for headloss as a function of flowrate for each pipe of the entire network was used to determine the respective pipe roughness factor (C). The relationship $h_L = Aq^B$ was used to represent the regression function $Y=a_0X^{a1}$ in order to determine the values for A and B. Then equation (2.4) was used to obtain the value for the respective pipe roughness factor C.

Table 4.10 presents the roughness factor for PIPE1 whose value is 117 as a whole number. The relationship for headloss and flow with respect to PIPE1 was determined as given by the equation (4.2) given as:

$$h_L = 13599.68q^{1.85} \tag{4.2}$$

Table 4.10: Regression Analysis of Flow Rate and Headloss for PIPE1

		$H_L= Aq^B(Y=a_0X^{a1})$ Where $A=a_0$ and $B=a_1$															
Y_i	**$\ln Y_i$**	**$(\ln Y_i)^2$**	**X_i**	**$\ln X_i$**	**$(\ln X_i)^2$**	**$\ln X_i \ln Y_i$**	**Sxx**	**Syy**	**Sxy**	**B**	**lnA**	**A**	**C^1.852**	**LOG C**	**C**	**R**	**R^2**
6.200	1.825	3.329	0.016	-4.157	17.284	-7.585	0.06	0.21	0.11	1.85	9.52	13599.68	6805.09	2.07	117.37	1.00	1.00
7.017	1.948	3.796	0.017	-4.090	16.732	-7.969											
7.995	2.079	4.321	0.018	-4.020	16.160	-8.357											
8.042	2.085	4.346	0.018	-4.017	16.135	-8.374											
8.148	2.098	4.401	0.018	-4.010	16.077	-8.412											
8.435	2.132	4.547	0.018	-3.991	15.928	-8.510											
8.553	2.146	4.607	0.019	-3.983	15.868	-8.550											
8.910	2.187	4.784	0.019	-3.961	15.693	-8.664											
8.997	2.197	4.826	0.019	-3.956	15.651	-8.691											
9.432	2.244	5.036	0.020	-3.931	15.450	-8.821											
9.852	2.288	5.233	0.020	-3.907	15.266	-8.938											
9.938	2.296	5.273	0.020	-3.902	15.229	-8.962											
101.52	**25.52**	**54.50**	**0.22**	**-47.93**	**191.47**	**-101.83**											

Table 4.11: Regression Analysis of Flow Rate and Headloss for PIPE 2

		$H_L = Aq^B (Y=a_0X^{a1})$ Where $A=a_0$ and $B=a_1$															
Yi	**lnYi**	**$(\ln Yi)^2$**	**Xi**	**lnXi**	**$(\ln Xi)^2$**	**lnXilnYi**	**Sxx**	**Syy**	**Sxy**	**B**	**lnA**	**A**	**C^1.852**	**LOG C**	**C**	**R**	**R^2**
6.94	1.94	3.75	0.00901	-4.71	22.18	-9.12	0.01	0.04	0.02	1.85	10.65	42398.24	7124.64	2.08	120	1.00	1.00
7.33	1.99	3.97	0.00928	-4.68	21.90	-9.32											
7.40	2.00	4.00	0.00932	-4.68	21.86	-9.35											
7.50	2.01	4.06	0.00939	-4.67	21.79	-9.40											
7.60	2.03	4.11	0.00946	-4.66	21.72	-9.45											
7.70	2.04	4.16	0.00952	-4.65	21.66	-9.50											
7.91	2.07	4.28	0.00966	-4.64	21.52	-9.59											
8.02	2.08	4.33	0.00974	-4.63	21.45	-9.64											
8.10	2.09	4.38	0.00979	-4.63	21.40	-9.68											
8.24	2.11	4.45	0.00988	-4.62	21.32	-9.74											
8.38	2.13	4.52	0.00997	-4.61	21.23	-9.80											
8.51	2.14	4.58	0.01005	-4.60	21.16	-9.85											
93.61	**24.63**	**50.60**	**0.12**	**-55.77**	**259.21**	**-114.45**											

Table 4.11 presents the roughness factor for PIPE2 whose value is 120. The relationship for headloss and flow with respect to PIPE2 was determined as given by the equation (4.3) given as:

$$h_L = 42398.24q^{1.85} \tag{4.3}$$

Table 4.12: Regression Analysis of Flow Rate and Headloss for PIPE 3

		$H_L = Aq^B (Y=a_0X^{a1})$ Where $A=a_0$ and $B=a_1$															
Y_i	**$\ln Y_i$**	**$(\ln Y_i)^2$**	**X_i**	**$\ln X_i$**	**$(\ln X_i)^2$**	**$\ln X_i \ln Y_i$**	**Sxx**	**Syy**	**Sxy**	**B**	**lnA**	**A**	**C^1.852**	**LOG C**	**C**	**R**	**R^2**
22.95	3.13	9.82	0.018	-4.02	16.17	-12.60	0.003	0.010	0.006	1.859	10.609	40480.76	6689.52	2.07	116	1	1
23.14	3.14	9.87	0.018	-4.02	16.14	-12.62											
23.35	3.15	9.93	0.018	-4.01	16.10	-12.64											
23.70	3.17	10.02	0.018	-4.00	16.04	-12.68											
23.79	3.17	10.04	0.018	-4.00	16.02	-12.68											
23.94	3.18	10.09	0.018	-4.00	15.99	-12.70											
24.42	3.20	10.21	0.019	-3.99	15.91	-12.74											
24.41	3.19	10.21	0.019	-3.99	15.91	-12.74											
24.57	3.20	10.25	0.019	-3.99	15.88	-12.76											
24.77	3.21	10.30	0.019	-3.98	15.85	-12.78											
24.96	3.22	10.35	0.019	-3.98	15.81	-12.79											
25.21	3.23	10.42	0.019	-3.97	15.77	-12.82											
289.2	**38.2**	**121.5**	**0.2**	**-47.9**	**191.6**	**-152.6**											

Table 4.12 presents the roughness factor for PIPE3 whose value is 116. The relationship for headloss and flow with respect to PIPE3 was determined as given by the equation (4.4) given as:

$$h_L = 40480.52q^{1.859}. \quad (4.4)$$

Table 4.13: Regression Analysis of Flow Rate and Headloss for PIPE 4

		$H_L = Aq^B (Y=a_0X^{a1})$ Where $A=a_0$ and $B=a_1$															
Y_i	**$\ln Y_i$**	**$(\ln Y_i)^2$**	**X_i**	**$\ln X_i$**	**$(\ln X_i)^2$**	**$\ln X_i \ln Y_i$**	**Sxx**	**Syy**	**Sxy**	**B**	**lnA**	**A**	**C^1.852**	**LOG C**	**C**	**R**	**R^2**
7.77	2.05	4.20	0.0068	-5.00	24.98	-10.24	0.02	0.077	0.04	1.86	11.35	84546.4	6811.9	2.07	117	1.0	1
7.99	2.08	4.32	0.0069	-4.98	24.82	-10.35											
8.07	2.09	4.36	0.0069	-4.98	24.77	-10.39											
8.15	2.10	4.40	0.0069	-4.97	24.72	-10.43											
8.26	2.11	4.46	0.0070	-4.96	24.65	-10.48											
8.69	2.16	4.68	0.0072	-4.94	24.38	-10.68											
8.76	2.17	4.71	0.0072	-4.93	24.34	-10.71											
9.06	2.20	4.86	0.0073	-4.91	24.16	-10.83											
9.37	2.24	5.00	0.0075	-4.90	23.98	-10.96											
9.64	2.27	5.13	0.0076	-4.88	23.83	-11.06											
9.69	2.27	5.16	0.0076	-4.88	23.80	-11.08											
9.85	2.29	5.23	0.0077	-4.87	23.72	-11.14											
105.28	**26.02**	**56.50**	**0.09**	**-59.21**	**292.14**	**-128.35**											

Table 4.13 presents the roughness factor for PIPE4 whose value is 117. The relationship for headloss and flow with respect to PIPE4 was determined as given by the equation (4.5) given as:

$$h_L = 84546.4q^{1.86} \tag{4.5}$$

Table 4.14: Regression Analysis of Flow Rate and Headloss for PIPE 5

		$H_L = Aq^B (Y=a_0X^{a1})$ Where $A=a_0$ and $B=a_1$															
Y_i	$\ln Y_i$	$(\ln Y_i)^2$	X_i	$\ln X_i$	$(\ln X_i)^2$	$\ln X_i \ln Y_i$	Sxx	Syy	Sxy	B	lnA	A	C^1.852	LOG C	C	R	R^2
1.89	0.64	0.40	0.0061	-5.10	26.05	-3.24	0.21	0.71	0.38	1.85	10.07	23663.04	7091.98	2.08	120.02	1.00	1.00
1.97	0.68	0.46	0.0062	-5.08	25.83	-3.44											
2.39	0.87	0.76	0.0069	-4.98	24.78	-4.34											
2.63	0.97	0.93	0.0073	-4.93	24.27	-4.75											
2.91	1.07	1.14	0.0077	-4.87	23.74	-5.20											
2.65	0.98	0.95	0.0073	-4.92	24.25	-4.81											
3.09	1.13	1.27	0.0079	-4.84	23.41	-5.46											
3.30	1.19	1.43	0.0082	-4.80	23.07	-5.73											
3.49	1.25	1.56	0.0085	-4.77	22.77	-5.96											
3.71	1.31	1.72	0.0087	-4.74	22.46	-6.22											
3.89	1.36	1.85	0.0090	-4.71	22.22	-6.40											
4.07	1.40	1.97	0.0092	-4.69	21.99	-6.58											
35.98	**12.84**	**14.44**	**0.09**	**-58.44**	**284.85**	**-62.14**											

Table 4.14 presents the roughness factor for PIPE5 whose value is 117. The relationship for headloss and flow with respect to PIPE5 was determined as given by the equation (4.6) given as:

$$h_L = 23663.04q^{1.85} \tag{4.6}$$

Table 4.15: Regression Analysis of Flow Rate and Headloss for PIPE 6

$H_L = Aq^B (Y=a_0X^{a1})$ Where $A=a_0$ and $B=a_1$

Y_i	$\ln Y_i$	$(\ln Y_i)^2$	X_i	$\ln X_i$	$(\ln X_i)^2$	$\ln X_i \ln Y_i$	Sxx	Syy	Sxy	B	lnA	A	C^1.852	LOG C	C	R	R^2
36.11	3.59	12.86	0.0162	-4.12	17.01	-14.79	0.0008	0.0027	0.0015	1.85	11.23	75209.19	6856.32	2.07	118	1.00	1.00
36.14	3.59	12.87	0.0162	-4.12	17.01	-14.80											
36.49	3.60	12.94	0.0163	-4.12	16.97	-14.82											
36.62	3.60	12.96	0.0163	-4.12	16.95	-14.82											
36.80	3.61	13.00	0.0163	-4.11	16.93	-14.84											
36.89	3.61	13.02	0.0164	-4.11	16.92	-14.84											
37.05	3.61	13.05	0.0164	-4.11	16.90	-14.85											
37.27	3.62	13.09	0.0164	-4.11	16.87	-14.86											
37.39	3.62	13.12	0.0165	-4.11	16.86	-14.87											
37.55	3.63	13.15	0.0165	-4.10	16.84	-14.88											
37.71	3.63	13.18	0.0165	-4.10	16.82	-14.89											
37.82	3.63	13.20	0.0166	-4.10	16.81	-14.89											
443.85	**43.33**	**156.43**	**0.20**	**-49.34**	**202.89**	**-178.15**											

Table 4.15 presents the roughness factor for PIPE6 whose value is 118. The relationship for headloss and flow with respect to PIPE6 was determined as given by the equation (4.6) given as:

$$h_L = 75209.19.4q^{1.85} \tag{4.6}$$

Table 4.16: Regression Analysis of Flow Rate and Headloss for PIPE 7

		H_L= Aq^B($Y=a_0X^{a1}$) Where A=a_0 and B=a_1															
Y_i	**lnY_i**	**$(lnY_i)^2$**	**X_i**	**lnX_i**	**$(lnX_i)^2$**	**lnX_ilnY_i**	**Sxx**	**Syy**	**Sxy**	**B**	**lnA**	**A**	**C^1.852**	**LOG C**	**C**	**R**	**R^2**
14.83	2.70	7.27	0.014	-4.24	18.00	-11.44	0.008	0.027	0.01	1.86	10.57	38880.91	6984.40	2.08	119.03	1.00	1.00
15.00	2.71	7.33	0.014	-4.24	17.94	-11.47											
15.07	2.71	7.36	0.014	-4.23	17.92	-11.48											
15.28	2.73	7.43	0.015	-4.23	17.86	-11.52											
15.39	2.73	7.47	0.015	-4.22	17.83	-11.54											
15.48	2.74	7.50	0.015	-4.22	17.80	-11.56											
16.16	2.78	7.74	0.015	-4.20	17.61	-11.68											
16.25	2.79	7.77	0.015	-4.19	17.58	-11.69											
16.48	2.80	7.85	0.015	-4.19	17.52	-11.73											
16.66	2.81	7.91	0.015	-4.18	17.47	-11.76											
16.87	2.83	7.98	0.015	-4.17	17.41	-11.79											
16.99	2.83	8.02	0.015	-4.17	17.38	-11.81											
190.46	**33.16**	**91.67**	**0.179**	**-50.47**	**212.32**	**-139.47**											

Table 4.16 presents the roughness factor for PIPE7 whose value is 119. The relationship for headloss and flow with respect to PIPE7 was determined as given by the equation (4.7) given as:

$$h_L = 38880.91q^{1.86} \quad (4.7)$$

Table 4.17: Regression Analysis of Flow Rate and Headloss for PIPE 8

		H_L= Aq^B($Y=a_0X^{a1}$) Where $A=a_0$ and $B=a_1$															
Yi	**lnYi**	**$(\ln Yi)^2$**	**Xi**	**lnXi**	**$(\ln Xi)^2$**	**lnXilnYi**	**Sxx**	**Syy**	**Sxy**	**B**	**lnA**	**A**	**C^1.852**	**LOG C**	**C**	**R**	**R^2**
21.55	3.07	9.43	0.0145	-4.23	17.93	-13.00	0.00	0.01	0.01	1.85	10.91	54693.35	7112.97	2.08	120.21	1.00	1.00
21.64	3.07	9.45	0.0145	-4.23	17.91	-13.01											
21.72	3.08	9.47	0.0145	-4.23	17.90	-13.02											
21.87	3.09	9.52	0.0146	-4.23	17.86	-13.04											
22.02	3.09	9.56	0.0147	-4.22	17.83	-13.06											
22.20	3.10	9.61	0.0147	-4.22	17.80	-13.08											
22.70	3.12	9.75	0.0149	-4.21	17.69	-13.13											
22.52	3.11	9.70	0.0148	-4.21	17.73	-13.11											
23.09	3.14	9.85	0.0150	-4.20	17.62	-13.18											
23.59	3.16	9.99	0.0152	-4.19	17.52	-13.23											
23.42	3.15	9.95	0.0152	-4.19	17.55	-13.21											
23.53	3.16	9.97	0.0152	-4.19	17.53	-13.22											
269.83	**37.35**	**116.25**	**0.18**	**-50.54**	**212.89**	**-157.30**											

Table 4.17 presents the roughness factor for PIPE8 whose value is 120. The relationship for headloss and flow with respect to PIPE8 was determined as given by the equation (4.8) given as:

$$h_L = 54693.354q^{1.85} \tag{4.8}$$

Table 4.18: Regression Analysis of Flow Rate and Headloss for PIPE 9

		$H_L = Aq^B (Y=a_0X^{a1})$ Where $A=a_0$ and $B=a_1$															
Yi	**lnYi**	**$(lnYi)^2$**	**Xi**	**lnXi**	**$(lnXi)^2$**	**lnXilnYi**	**Sxx**	**Syy**	**Sxy**	**B**	**lnA**	**A**	**C^1.852**	**LOG C**	**C**	**R**	**R^2**
3.08	1.12	1.27	0.00410	-5.50	30.21	-6.18	0.10	0.33	0.18	1.85	11.31	81743.24	7036.15	2.08	120	1.00	1.00
3.26	1.18	1.40	0.00423	-5.47	29.87	-6.46											
3.35	1.21	1.46	0.00429	-5.45	29.71	-6.59											
3.60	1.28	1.64	0.00446	-5.41	29.29	-6.93											
3.96	1.38	1.89	0.00470	-5.36	28.74	-7.37											
4.06	1.40	1.96	0.00476	-5.35	28.59	-7.49											
4.18	1.43	2.04	0.00483	-5.33	28.43	-7.62											
4.38	1.48	2.18	0.00496	-5.31	28.15	-7.84											
4.66	1.54	2.37	0.00513	-5.27	27.80	-8.12											
4.84	1.58	2.48	0.00523	-5.25	27.59	-8.28											
4.96	1.60	2.56	0.00531	-5.24	27.45	-8.39											
5.17	1.64	2.70	0.00542	-5.22	27.22	-8.57											
49.49	**16.84**	**23.96**	**0.0574**	**-64.15**	**343.05**	**-89.84**											

Table 4.18 presents the roughness factor for PIPE9 whose value is 120. The relationship for headloss and flow with respect to PIPE9 was determined as given by the equation (4.9) given as:

$$h_L = 81743.24.4q^{1.85} \quad (4.9)$$

Table 4.19: Updated Roughness Factors (C) for Maralal Water Distribution Network

PIPE ID	LENGTH (m)	DIAMETER (mm)	DETERMINED ROUGHNESS(C)	VARIATION FROM 140 (%)
PIPE0	2200	200	65.5	-53
PIPE1	874	150	117	-16
PIPE2	396	100	120	-14
PIPE3	355	100	116	-17
PIPE4	755	100	117	-16
PIPE5	220	100	120	-14
PIPE6	676	100	118	-15.7
PIPE7	356	100	119	-15
PIPE8	510	100	120	-14
PIPE9	754	100	120	-14

From Table 4.19, the current roughness factors for the respective pipe diameters of Maralal distribution network were determined. The determined roughness factors (C) for PIPE0, PIPE1, PIPE2, PIPE3, PIPE4, PIPE5, PIPE6, PIPE7, PIPE8 and PIPE9 are 65.5, 117, 120, 116, 117, 120, 118, 119, 120 and 120 respectively. A lower roughness factor indicates a more roughness and vice versa. PIPE0 has the lowest value of roughness factor of 65.5 because it is the main distribution pipe which despite zonal water rationing conveys water daily hence more chlorine decay that causes biofilm formation on the pipe walls. This in turn contributes to increased pipe roughness thus lower roughness factor. It is observed that roughness factor (%) for PIPE0, PIPE1, PIPE2, PIPE3, PIPE4, PIPE5, PIPE6, PIPE7, PIPE8 and PIPE9 varies by 53, 16, 14, 17, 16, 14, 15.7, 15, 14 and 14 respectively. These results are similar to study results by Shashikumar *et al.* (2003), who concluded from their field observation that roughness factors in pipes can vary by over 25%.

Table 4.20: Percentage Increase in Headloss (m) due to Change in C- Factor

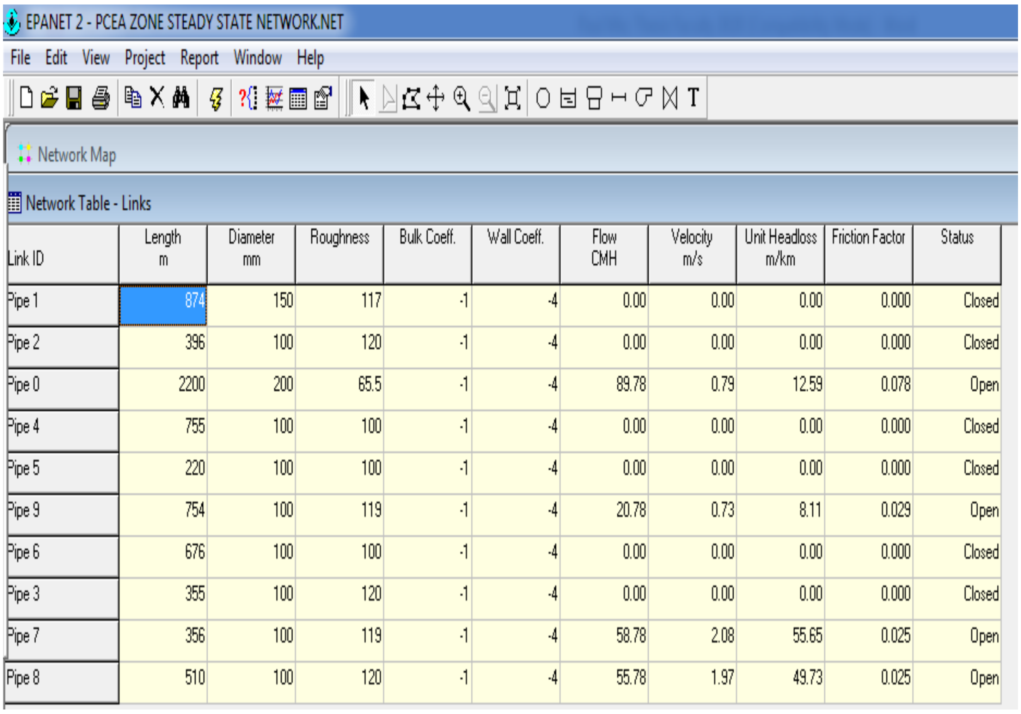
PIPE ID	CURRENT PIPE-C-FACTOR	NEW PIPE C-FACTOR	CURRENT PIPE HEADLOSS	ORIGINAL PIPE HEADLOSS	HEADLOSS INCREASE (%)
PIPE0	65.5	140	27.19	6.7	306
PIPE1	117	140	14.9	10.7	39
PIPE2	120	140	8.1	6.1	33
PIPE3	116	140	25.4	17.9	42
PIPE4	117	140	9.8	7.0	40
PIPE5	120	140	3.0	2.3	30
PIPE6	118	140	39.0	28.4	37
PIPE7	119	140	17.1	12.7	35
PIPE8	120	140	23.3	17.5	33
PIPE9	120	140	4.4	3.2	38

From the results presented in Table 4.20, it is observed that increase in roughness factor causes a corresponding increase in headloss. The percentage increase in headloss for PIPE0, PIPE1, PIPE2, PIPE3, PIPE4, PIPE5, PIPE6, PIPE7, PIPE8 and PIPE9 are 306, 39, 33, 42, 40, 30, 37, 35, 33 and 38m respectively. These results are in line with Feinauer *et al.* (2008) who found that the change in roughness factor causes the head loss to increase by nearly 45%.

4.2 Simulation and Prediction of the Steady and Dynamic States Hydraulic Parameters

4.2.1 Model Calibration for Steady Simulation

The simulation was carried on each zone and the results for the hydraulic parameters along pipes and at nodes were obtained and presented a network tables as presented in Appendix B. Pipes are referred to as Links by the model. Output results generated for pipes (links) are flow rate and head loss, while output results generated for nodes are the pressure values. Figure 4.5 presents a summary of PCEA zone steady state pipe network results.



EPANET 2 - PCEA ZONE STEADY STATE NETWORK.NET

File Edit View Project Report Window Help

Network Map

Network Table - Links

Link ID	Length m	Diameter mm	Roughness	Bulk Coeff.	Wall Coeff.	Flow CMH	Velocity m/s	Unit Headloss m/km	Friction Factor	Status
Pipe 1	874	150	117	-1	-4	0.00	0.00	0.00	0.000	Closed
Pipe 2	396	100	120	-1	-4	0.00	0.00	0.00	0.000	Closed
Pipe 0	2200	200	65.5	-1	-4	89.78	0.79	12.59	0.078	Open
Pipe 4	755	100	100	-1	-4	0.00	0.00	0.00	0.000	Closed
Pipe 5	220	100	100	-1	-4	0.00	0.00	0.00	0.000	Closed
Pipe 9	754	100	119	-1	-4	20.78	0.73	8.11	0.029	Open
Pipe 6	676	100	100	-1	-4	0.00	0.00	0.00	0.000	Closed
Pipe 3	355	100	120	-1	-4	0.00	0.00	0.00	0.000	Closed
Pipe 7	356	100	119	-1	-4	58.78	2.08	55.65	0.025	Open
Pipe 8	510	100	120	-1	-4	55.78	1.97	49.73	0.025	Open

Figure 4.5: PCEA Zone Steady State Pipe Network

From Figure 4.5, only PCEA zone pipes are open and indicate results for flow and headloss. Pipe 0, Pipe 7, Pipe 8 and Pipe 9 flow values were 89.78 m^3/hr, 58.78 m^3/hr, 55.78 m^3/hr and 20.78m^3/hr respectively. Flow results were simulated after selected node demands were inserted into the model in order to closely match the simulated flow with field flow values. Unit Headloss values for Pipe 0, Pipe 7, Pipe 8 and Pipe 9 were multiplied by the respective pipe lengths to obtain the pipe headloss.

The rest of the steady state simulation results can be found in Figure B1 in Appendix B.

Tables 4.21, 4.22, 4.23 and 4.24 presented measured and simulated results for County zone, PCEA zone, Town zone 1 and Town zone 2 respectively. From the tables measured and simulated flow, measured and simulated headloss and nodal demands were presented for the respective zonal pipes and nodes.

Table 4.21: County Zone Measured and Simulated Data

MEASURED FLOW(m^3/hr)		MEASURED HEADLOSS(m)		SIMULATED FLOW(m^3/hr)		SIMULATED HEADLOSS(m)		NODE DEMANDS(m^3/hr)		
PIPE 1	PIPE 2	PIPE 1	PIPE 2	PIPE 1	PIPE 2	PIPE 1	PIPE 2	NODE 2	NODE 3	NODE 4
56.33	30.03	6.20	6.02	56.35	30.05	6.44	6.26	10.00	26.30	30.05
60.23	30.86	7.02	6.33	60.24	30.87	7.29	6.58	10.00	29.37	30.87
64.64	31.02	8.00	6.40	64.65	31.00	8.30	6.63	10.00	33.65	31.00
64.84	31.29	8.04	6.50	64.85	31.30	8.35	6.76	10.00	33.55	31.30
65.30	31.56	8.15	6.60	65.31	31.57	8.46	6.86	10.00	33.74	31.57
66.53	31.76	8.44	6.68	66.54	31.77	8.77	6.94	10.00	34.77	31.77
67.03	32.26	8.55	6.88	67.05	32.30	8.89	7.16	10.00	34.75	32.30
68.53	32.53	8.91	6.98	68.54	32.55	9.26	7.26	10.00	35.99	32.55
68.89	32.74	9.00	7.06	68.90	32.75	9.34	7.35	10.00	32.75	32.45
70.67	32.91	9.43	7.13	70.68	32.95	9.80	7.43	10.00	37.73	32.35
72.35	33.04	9.85	7.19	72.36	33.00	10.23	7.45	10.00	39.36	32.70
72.69	33.21	9.94	7.26	72.70	33.20	10.32	7.53	10.00	39.50	32.90

Table 4.22: PCEA Zone Measured and Simulated Data

MEASURED FLOW(m^3/hr)			MEASURED HEADLOSS(m)			SIMULATED FLOW(m^3/hr)			SIMULATED HEADLOSS(m)			NODE DEMANDS(m^3/hr)			
PIPE 7	PIPE 8	PIPE 9	PIPE 7	PIPE 8	PIPE 9	PIPE 7	PIPE 8	PIPE 9	PIPE 7	PIPE 8	PIPE 9	NODE 2	NODE 6	NODE 7	NODE 8
51.75	51.28	14.77	15.04	21.80	3.08	51.76	51.20	14.78	15.65	21.64	3.26	16.00	0.56	36.42	14.78
52.07	51.43	15.23	15.15	22.01	3.19	52.00	51.41	15.25	15.79	21.80	3.45	16.00	0.59	36.16	15.25
52.20	51.60	15.46	15.18	22.05	3.26	52.22	51.60	15.47	15.91	21.96	3.54	16.00	0.62	36.13	15.47
52.60	51.80	16.06	15.32	22.20	3.62	52.61	51.85	16.05	16.33	22.15	3.79	16.00	0.76	35.80	16.05
52.79	51.89	16.91	15.44	22.34	3.78	52.80	51.88	16.92	16.24	22.17	4.18	16.00	0.72	34.96	16.92
52.96	52.36	17.14	15.54	22.47	4.06	53.00	52.34	17.15	16.35	22.54	4.28	16.00	0.66	35.19	17.15
54.20	53.71	17.41	16.66	22.73	4.14	54.19	53.69	17.42	17.04	23.63	4.41	16.00	0.50	36.27	17.42
54.37	53.92	17.86	16.70	22.87	4.94	54.35	53.91	17.85	17.13	23.81	4.61	16.00	0.44	36.06	17.85
54.78	53.95	18.47	16.82	23.14	4.99	54.75	53.96	18.45	17.37	23.85	4.91	16.00	0.79	35.51	18.45
55.10	54.67	18.84	16.90	23.82	5.15	55.12	54.52	18.86	17.59	24.31	5.11	16.00	0.60	35.66	18.86
55.48	55.39	19.10	17.22	24.08	5.23	55.50	55.36	19.10	17.81	25.01	5.23	16.00	0.14	36.26	18.70
55.69	55.46	19.68	17.34	24.28	5.41	55.70	55.45	19.78	17.93	25.08	5.58	16.00	0.25	35.67	19.30

Table 4.23: Town Zone 1 Measured and Simulated Data

MEASURED FLOW(m^3/hr)			MEASURED HEADLOSS(m)			SIMULATED FLOW(m^3/hr)			SIMULATED HEADLOSS(m)			NODE DEMANDS(m^3/hr)			
PIPE 1	PIPE 3	PIPE 4	PIPE 1	PIPE 3	PIPE 4	PIPE 1	PIPE 3	PIPE 4	PIPE 1	PIPE 3	PIPE 4	NODE 2	NODE 3	NODE 5	NODE 9
82.25	64.52	24.69	12.95	24.00	7.99	82.25	64.55	24.55	12.98	24.63	8.61	5.00	17.70	40.00	24.55
82.94	64.80	24.86	13.26	24.55	8.09	83.23	64.78	25.48	13.26	24.79	9.22	5.00	18.45	39.30	25.48
83.72	65.12	24.91	13.47	24.65	8.12	83.97	65.10	25.90	13.48	25.02	9.50	5.00	18.87	39.20	25.90
85.63	65.63	25.42	14.22	25.24	8.43	84.40	65.67	26.50	13.61	25.43	9.91	5.00	18.73	39.17	26.50
86.76	65.77	26.43	14.33	25.40	9.06	85.24	65.80	26.60	13.86	25.52	9.98	5.00	19.44	39.20	26.60
86.88	66.00	26.94	14.37	25.68	9.39	85.67	66.05	26.80	13.99	25.70	10.12	5.00	19.62	39.25	26.80
87.12	66.70	27.08	14.49	26.24	9.48	86.19	66.68	27.60	14.15	26.15	10.69	5.00	19.51	39.08	27.60
87.44	66.69	27.25	14.57	26.35	9.59	87.07	66.72	28.55	14.42	26.92	11.39	5.00	19.35	39.17	28.55
87.79	66.92	27.57	14.68	26.54	9.80	87.87	66.98	29.20	14.67	26.38	11.87	5.00	20.89	37.78	29.20
88.18	67.21	28.28	14.79	26.67	10.27	88.39	67.20	29.52	14.82	26.54	12.11	5.00	21.19	37.68	29.52
88.89	67.50	28.34	14.82	27.01	10.31	88.87	67.55	30.00	14.97	26.80	12.47	5.00	21.32	37.55	30.00
89.50	67.86	28.44	15.06	27.23	10.38	89.49	67.90	30.45	15.17	27.05	12.83	5.00	21.59	37.45	30.45

Table 4.24: Town Zone 2 Measured and Simulated Data

MEASURED FLOW(m^3/hr)			MEASURED HEADLOSS(m)			SIMULATED FLOW(m^3/hr)			SIMULATED HEADLOSS(m)			NODE DEMANDS(m^3/hr)			
PIPE 1	PIPE 6	PIPE 5	PIPE 1	PIPE 6	PIPE 5	PIPE 1	PIPE 6	PIPE 5	PIPE 1	PIPE 6	PIPE 5	NODE 2	NODE 3	NODE 7	NODE 9
89.67	58.20	21.86	14.66	36.31	1.89	89.65	58.20	21.82	15.22	37.52	1.92	2.00	31.45	36.38	21.82
90.27	58.23	22.35	14.84	36.73	1.97	90.20	58.22	22.35	15.39	37.54	2.01	2.00	31.98	35.87	22.35
91.32	58.53	24.80	15.17	36.77	2.39	91.30	58.51	24.82	15.74	37.89	2.44	2.00	32.79	33.69	24.82
91.51	58.64	26.10	15.22	36.85	2.63	91.50	58.60	26.08	15.80	38.00	2.68	2.00	32.90	32.52	26.08
92.00	58.80	27.56	15.37	36.98	2.91	92.00	58.75	27.50	15.97	38.17	2.95	2.00	33.25	31.25	27.50
92.52	58.88	28.12	15.54	37.07	2.65	92.45	58.85	28.10	16.11	38.30	3.07	2.00	33.60	30.75	28.10
92.98	59.02	28.50	15.68	37.29	3.09	93.00	59.00	28.52	16.29	38.48	3.16	2.00	34.00	30.48	28.52
93.40	59.21	29.53	15.81	37.56	3.30	93.35	59.20	29.50	16.40	38.72	3.36	2.00	34.15	29.70	29.50
93.66	59.31	30.46	15.89	37.60	3.49	93.60	59.30	30.45	16.48	38.84	3.57	2.00	34.30	28.85	30.45
94.00	59.44	31.41	16.00	37.74	3.71	94.05	59.45	31.40	16.63	39.03	3.78	2.00	34.60	28.05	31.40
94.45	59.58	32.31	16.14	37.99	3.89	94.40	59.60	32.30	16.75	39.21	3.98	2.00	34.80	27.30	32.30
95.48	59.67	33.10	16.47	38.36	4.07	95.50	59.65	33.10	17.11	39.27	4.16	2.00	35.85	26.55	33.10

4.2.2 Model Calibration for Dynamic Simulation

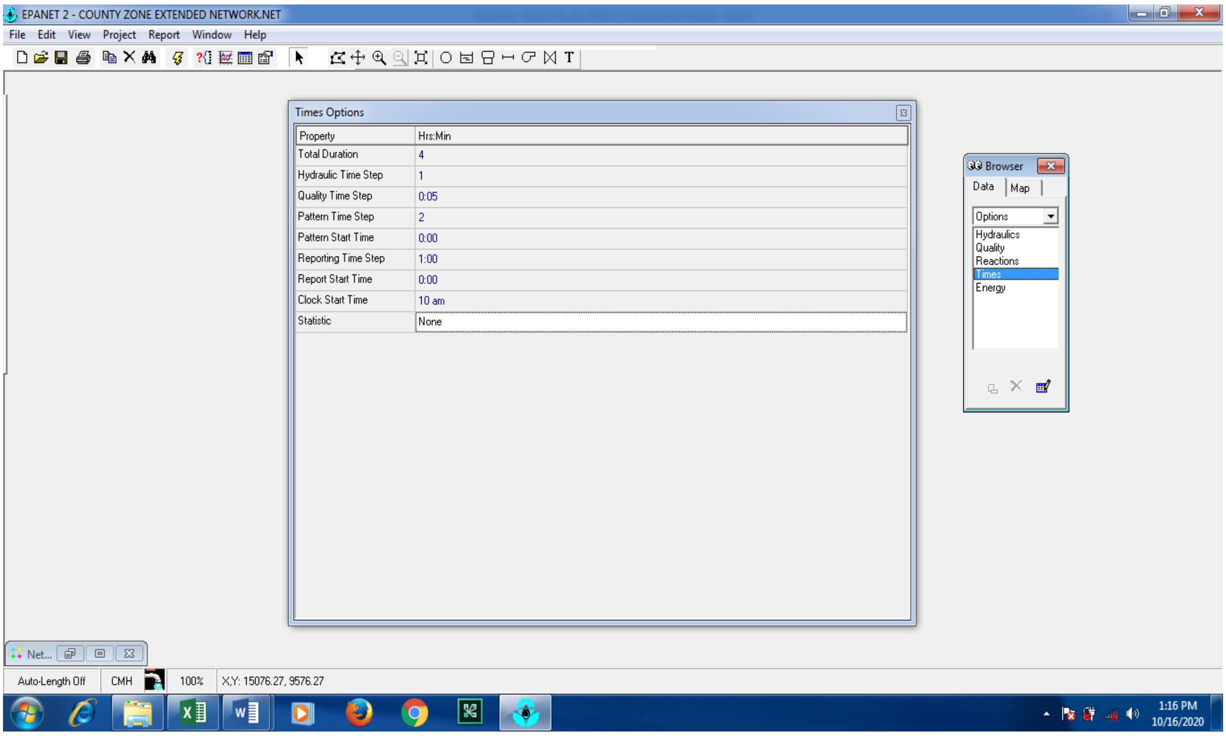


Figure 4.6: Hydraulic and Pattern Time Step for County Zone Extended Period Network

From Figure 4.6, a pattern of 2-hour step was used thus making demands to be changed at two different patterns of 4-hour period each with a 1-hr hydraulic time step.

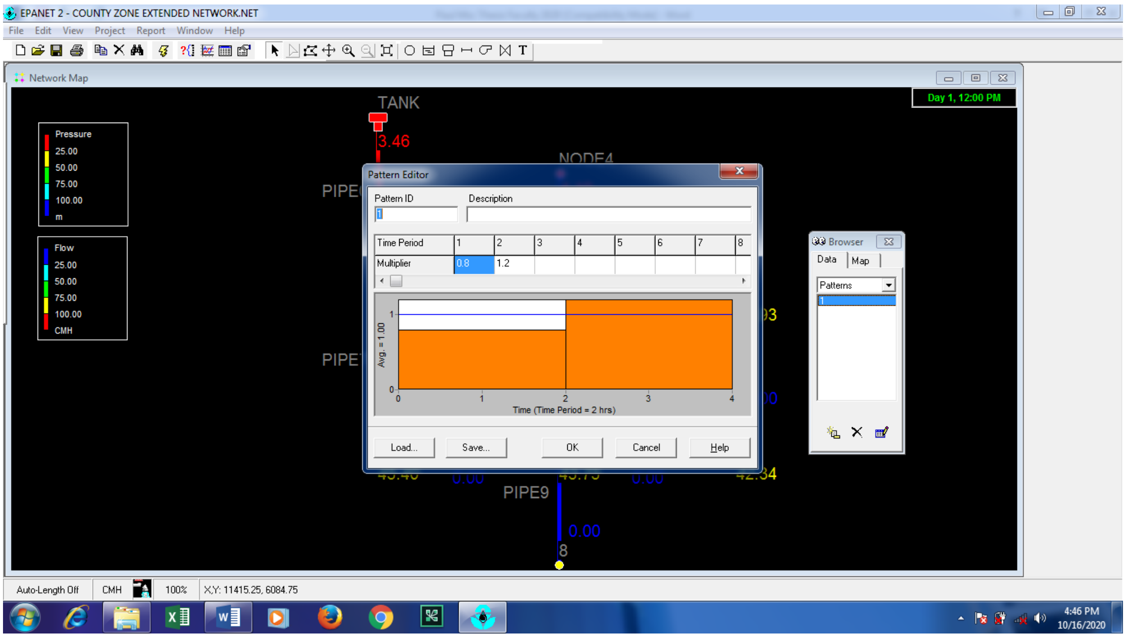


Figure 4.7: Time Pattern Multipliers for County Zone

Figure 4.7, shows a time pattern where for time periods 1 and 2 multipliers of 0.8 and 1.2 have been assigned respectively. A time pattern interval has been set to 2 hours to give a total duration of 4 hours. During simulations, each pattern Node base demand was multiplied by the respective multiplier.

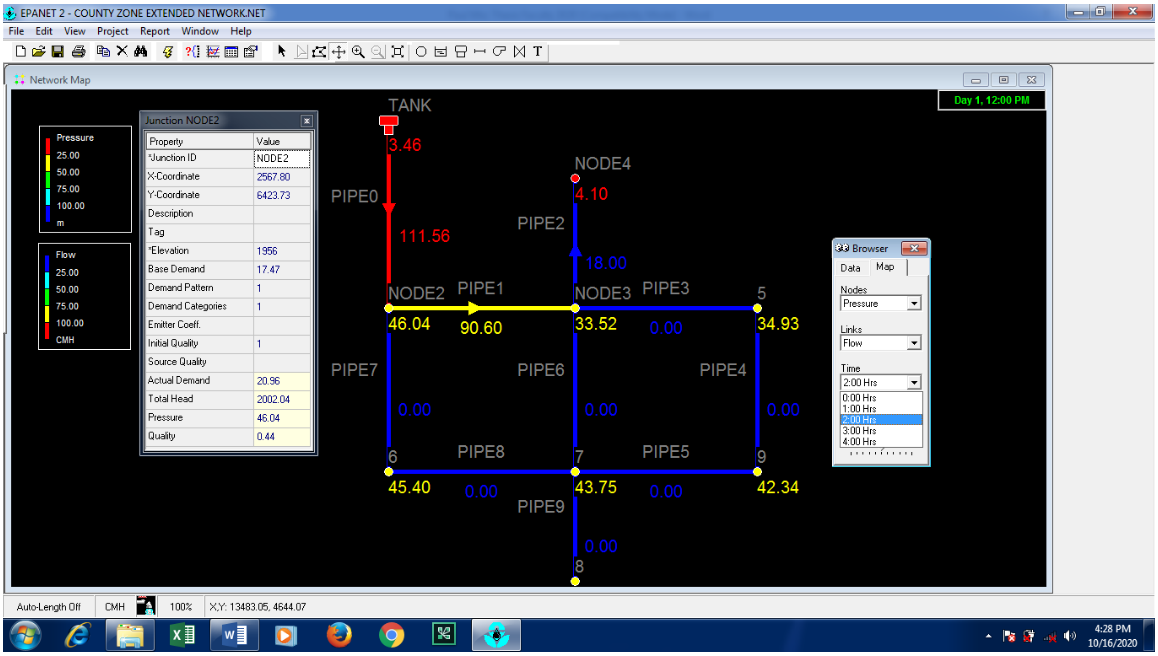


Figure 4.8: County Zone Extended Period Network Simulated Flow and Pressure (2-hr time step)

From Figure 4.8 base demands were multiplied by 1.2 where NODE2 base demand of 17.47m^3/hr was multiplied by 1.2 to obtain 20.96m^3/hr. The pressure values for Tank, NODE2, NODE3 and NODE4 are 3.46m, 46.06m, 33.52m and 4.10m respectively while flow rate results for PIPE0, PIPE1 and PIPE3 are 111.56 m^3/hr, 90.6 m^3/hr and 18 m^3/hr respectively. The rest of the dynamic state simulation results can be found in Figures B2 and B3 in Appendix B.

From Figure 4.9 the county zone nodes base demands were multiplied by 0.8 to obtain the dynamic (actual) state demand. The nodes were identified by the model as Jcn 2, Jcn 3, and Jcn 4 with base and actual demands as 17.47m^3/hr, 60.5 m^3/hr, 15 m^3/hr and 13.98 m^3/hr, 48.40 m^3/hr and 12 m^3/hr respectively. From Table 4.10 the unit headloss for PIPE0, PIPE1 and PIPE2 were 8.89m/km, 8.38m/km and 2.38m/km respectively. PIPE0 has a higher value of unit headloss due to a lower value of roughness factor of 65.5 while PIPE2 has a lower value of unit headloss due to a higher value of roughness factor which was 120.

Network Table - Nodes at 0:00 Hrs

Node ID	Elevation m	Base Demand CMH	Initial Quality mg/L	Demand CMH	Head m	Pressure m	Chlorine mg/L
Junc 2	1956	17.47	1	13.98	2024.65	68.65	1.00
Junc 3	1953	60.5	1	48.40	2017.33	64.33	1.00
Junc 4	1980	015	1	12.00	2016.18	36.18	1.00

Figure 4.9: County Zone Nodes Model Results for dynamic state

Network Table - Links at 0:00 Hrs

Link ID	Length m	Diameter mm	Roughness	Bulk Coeff.	Wall Coeff.	Flow CMH	Velocity m/s	Unit Headloss m/km	Friction Factor	Reaction Rate mg/L/d	Chlorine mg/L	Status
Pipe PIPE1	874	150	117	-1.0	-1.55	60.40	0.95	8.38	0.027	0.00	1.00	Open
Pipe PIPE2	396	100	120	-1.0	-1.55	12.00	0.42	2.89	0.031	0.00	1.00	Open
Pipe PIPE0	2200	200	65.5	-1	-1.55	74.38	0.66	8.89	0.081	0.00	1.00	Open

Figure 4.10: County Zone Pipes Model Results for dynamic state

From Figure 4.11 the PCEA zone nodes base demands were multiplied by 0.9 to obtain the dynamic (actual) state demand. The nodes were identified by the model as Jcn 2, Jcn 6, Jcn 7 and Jcn 8 with base and actual demands as 21.26m^3/hr, 11.51 m^3/hr, 9.82 m^3/hr, 37.06 m^3/hr and 19.13 m^3/hr, 10.36 m^3/hr, 8.84 m^3/hr and 33.35 m^3/hr respectively. From Table 4.12 the unit headloss for PIPE7, PIPE8 and PIPE9 were 45.22m/km, 29.65m/km and 19.49m/km respectively. PIPE7 has a higher value of unit headloss due to a higher value of velocity of 1.86m/s while PIPE9 has a lower value of unit headloss due to a lower value of velocity which was 1.18m/s.

Network Table - Nodes at 0:00 Hrs

Node ID	Elevation m	Base Demand CMH	Initial Quality mg/L	Demand CMH	Head m	Pressure m	CHLORINE mg/L
Junc 2	1956	21.26	1	19.13	2025.94	69.94	1.00
Junc 6	1951	11.51	1	10.36	2008.79	57.79	1.00
Junc 7	1947	9.82	1	8.84	1992.16	45.16	1.00
Junc 8	1945	37.06	1	33.35	1976.76	31.76	1.00

Figure 4.11: PCEA Zone Model Output for Nodes for dynamic state

Network Table - Links at 0:00 Hrs

Link ID	Length m	Diameter mm	Roughness	Bulk Coeff.	Wall Coeff.	Flow CMH	Velocity m/s	Unit Headloss m/km	Friction Factor	Reaction Rate mg/L/d	CHLORINE mg/L	Status
Pipe PIPE9	754	100	119	-1.0	-1.55	33.35	1.18	19.49	0.027	0.00	1.00	Open
Pipe PIPE7	356	100	119	-1.0	-1.55	52.55	1.86	45.22	0.026	0.00	1.00	Open
Pipe PIPE8	510	100	120	-1.0	-1.55	42.19	1.49	29.65	0.026	0.00	1.00	Open

Figure 4.12: PCEA Pipes Model Results for dynamic state

From Figure 4.13 the Town zone 1 nodes base demands were multiplied by 0.9 to obtain the dynamic (actual) state demand. The nodes were identified by the model as Jcn 2, Jcn 3, Jcn 5 and Jcn 9 with base and actual demands as 8.12m^3/hr, 45.68 m^3/hr, 21.33m^3/hr, 24.58 m^3/hr and 7.31 m^3/hr, 41.11 m^3/hr, 19.20 m^3/hr and 22.12 m^3/hr respectively. From Table 4.14 the unit headloss for PIPE0, PIPE1 and PIPE4 were 12.58m/km, 14.91m/km and 9.4m/km respectively. PIPE1 has a higher value of unit headloss due to a higher value of velocity of 1.3m/s while PIPE4 has a lower value of unit headloss due to a lower value of velocity of 0.78m/s.

Network Table - Nodes at 0:00 Hrs

Node ID	Elevation m	Base Demand CMH	Initial Quality mg/L	Demand CMH	Head m	Pressure m	Chlorine mg/L
Junc 9	1945	24.58	1	22.12	1986.28	41.28	1.00
Junc 3	1953	45.68	1	41.11	2003.50	50.50	1.00
Junc 5	1955	21.33	1	19.20	1993.05	38.05	1.00
Junc 2	1956	8.12	1	7.31	2016.52	60.52	1.00

Figure 4.13: Town Zone 1 Nodes Model Results for dynamic state

Network Table - Links at 0:00 Hrs

Link ID	Length m	Diameter mm	Roughness	Bulk Coeff.	Wall Coeff.	Flow CMH	Velocity m/s	Unit Headloss m/km	Friction Factor	Reaction Rate mg/L/d	Chlorine mg/L	Status
Pipe PIPE1	874	150	117	-1	-1.55	82.43	1.30	14.91	0.026	0.00	1.00	Open
Pipe PIPE0	2200	200	65.5	-1	-1.55	89.74	0.79	12.58	0.078	0.00	1.00	Open
Pipe PIPE4	755	100	117	-1	-1.55	22.12	0.78	9.40	0.030	0.00	1.00	Open

Figure 4.14: Town Zone 1 Pipes Model Results for dynamic state

From Figure 4.15 the Town zone 2 nodes base demands were multiplied by 0.9 to obtain the dynamic (actual) state demand. The nodes were identified by the model as Jcn 2, Jcn 3, Jcn 7, and Jcn 9 with base and actual demands as 1m^3/hr, 60.68 m^3/hr, 4.42 m^3/hr, 32.52 m^3/hr and 0.9 m^3/hr, 54.61 m^3/hr, 4.07 and 29.09m^3/hr respectively. From Table 4.16 the unit headloss for PIPE0, PIPE1, PIPE5 and PIPE6 were 12.30m/km, 16.74m/km, 14.89 m^3/hr and 19.58m/km respectively. PIPE0 has a lower value of unit headloss due to a lower value of roughness factor of 65.5 as well as lower value of velocity of 0.78m/s while PIPE6 has a higher value of unit headloss due to a higher value of roughness factor which was 118 as well as a higher value of velocity (1.17m/s).

Network Table - Nodes at 0:00 Hrs

Node ID	Elevation m	Base Demand CMH	Initial Quality mg/L	Demand CMH	Head m	Pressure m	Chlorine mg/L
Junc 2	1956	1	1	0.90	2017.13	61.13	1.00
Junc 3	1953	60.68	1	54.61	2002.50	49.50	1.00
Junc 7	1947	4.52	1	4.07	1989.27	42.27	1.00
Junc 9	1945	32.32	1	29.09	1985.89	40.89	1.00

Figure 4.15: Town Zone 2 nodes model results for dynamic state

Network Table - Links at 0:00 Hrs

Link ID	Length m	Diameter mm	Roughness	Bulk Coeff.	Wall Coeff.	Flow CMH	Velocity m/s	Unit Headloss m/km	Friction Factor	Reaction Rate mg/L/d	Chlorine mg/L	Status
Pipe PIPE5	220	100	120	-1	-1.15	29.09	1.03	14.89	0.028	0.00	1.00	Open
Pipe PIPE6	676	100	118	-1	-1.55	33.16	1.17	19.58	0.028	0.00	1.00	Open
Pipe PIPE1	874	150	117	-1	-1.55	87.77	1.38	16.74	0.026	0.00	1.00	Open
Pipe PIPE0	2200	200	65.5	-1	-1.55	88.67	0.78	12.30	0.079	0.00	1.00	Open

Figure 4.16: Town Zone 2 Pipes Model Results for dynamic state

4.2.3 Model Calibration Results

The output results of the model in regard to flow rate and head loss were compared with the measured data from the field as presented in Table 4.25.

Table 4.25: Nash-Sutcliffe Model Efficiency Coefficient (E) for PIPE5 Flow rate (X)

Xobs	Xmodel	Xobs-Xmodel	$(Xobs\text{-}Xmodel)^2$	Xobs-28	$(Xobs\text{-} 28)^2$
21.86	21.82	0.04	0.0016	-6.1483	37.802
22.35	22.35	0.00	0.0000	22.3500	499.523
24.80	24.82	-0.02	0.0004	24.800	615.040
26.10	26.08	0.02	0.0004	26.100	681.210
27.56	27.50	0.06	0.0036	27.5600	759.554
28.12	28.10	0.02	0.0004	28.1200	790.734
28.50	28.52	-0.02	0.0004	28.500	812.250
29.53	29.50	0.03	0.0009	29.5300	872.021
30.46	30.45	0.01	0.0001	30.4600	927.812
31.41	31.40	0.01	0.0001	31.4100	986.588
32.31	32.30	0.01	0.0001	32.3100	1043.940
33.10	33.10	0.00	0.0000	33.1000	1095.610
Av.28.0083			**0.0080**		**9122.079**

From Table 4.25 it can also be observed that the value of E=1-(0.008/9122.079) which equals to 0.999999 or 1. Since Nash-Sutcliffe efficiencies can range from -∞ to 1, an efficiency of 1 (E = 1) was used to correspond to a perfect match between model and observations.

4.2.4 Model Validation

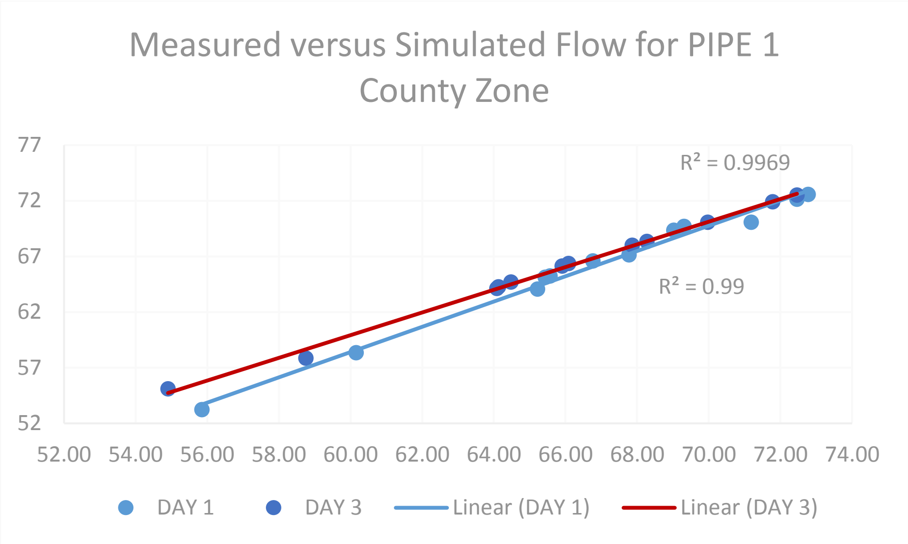


Figure 4.17: Measured Versus Simulated Flow for Day 1 and Day 3 of PIPE 1 in County Zone

From Figure 4.17, it can be observed that the coefficient of correlation between measured and simulated flowrate for Day 1 data set is 0.99 and for Day 3 data set is 0.9969. This is an indication that the model was validated.

4.2.5 Measured Versus Simulated Flow and Headloss for County Zone Pipes

From Figures 4.18 and 4.19 it is observed that the simulated flow results closely match the measured flow results with coefficients of determination (R^2) of 0.9918 and 0.9994 respectively. Higher values of R^2 indicate a less error variance and typically values greater than 0.5 are considered acceptable (Santhi *et al.*, 2001).

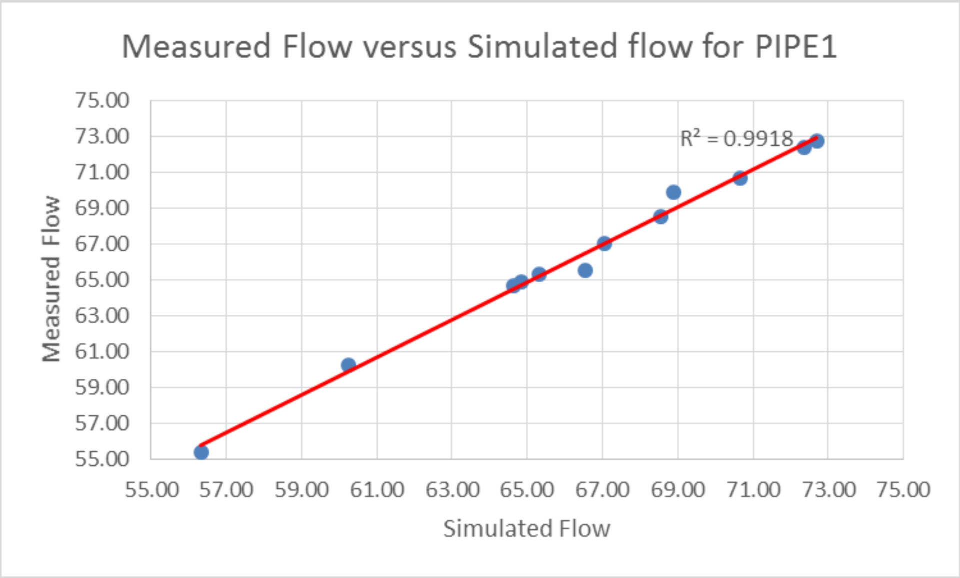


Figure 4.18: Measured Vs. Simulated flow for Pipe 1 in County zone

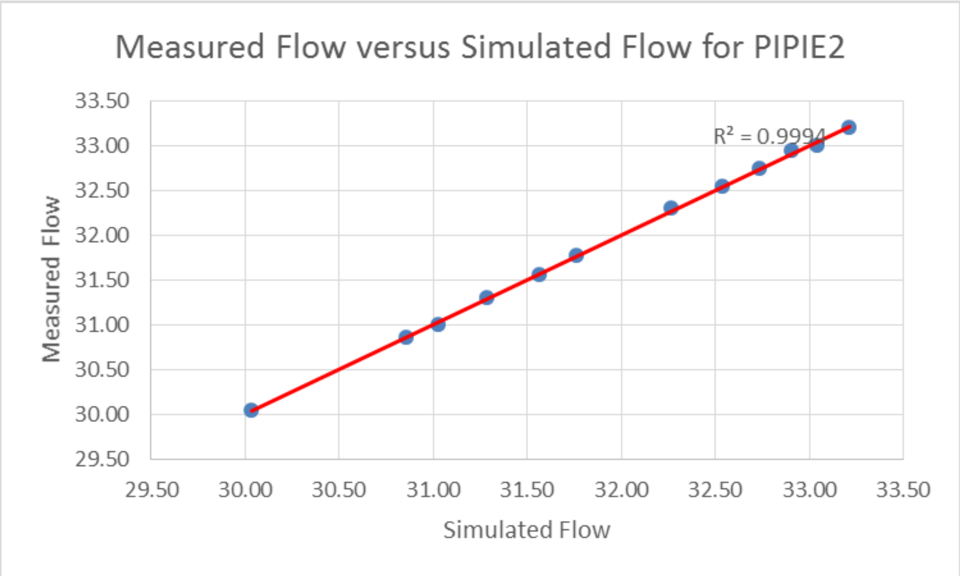


Figure 4.19: Measured versus Simulated flow for Pipe 2 in County zone

From Figures 4.20 and 4.21 results indicates that the simulated headloss results closely match the calculated headloss results with coefficients of determination (R^2) of 0.9978 and 1 respectively. Henseler (2009) proposed a rule of thumb for acceptable R^2 with 0.75, 0.50 and 0.25 are described as substantial, moderate and weak respectively.

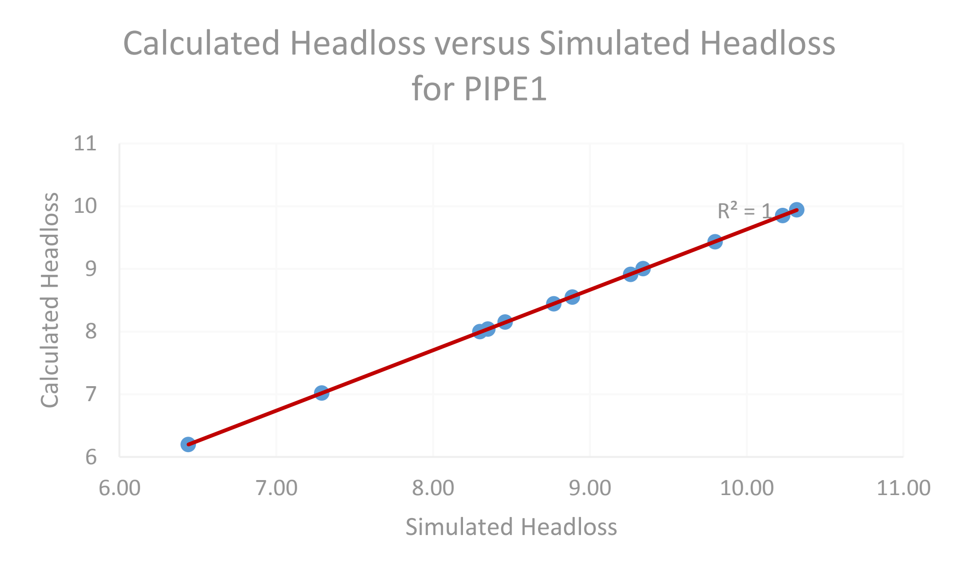


Figure 4.20: Calculated versus simulated headloss for Pipe 1 in County zone

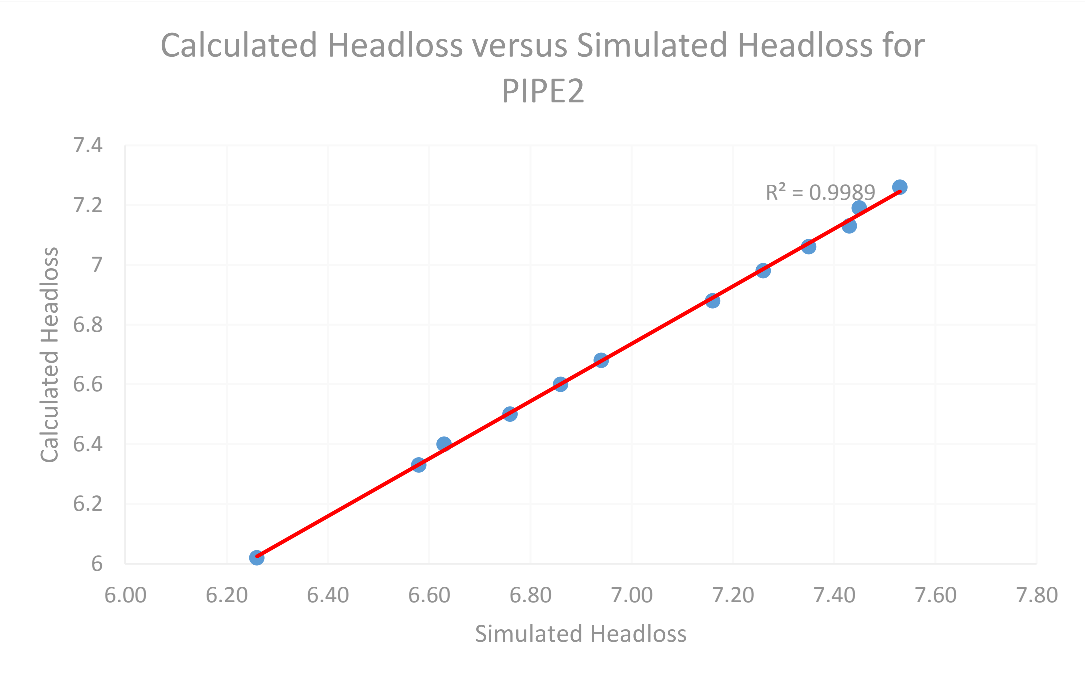


Figure 4.21: Calculated versus simulated headloss for Pipe 2 in County zone

4.2.6 Measured Versus Simulated Flow and Headloss for PCEA Zone Pipes

From Figures 4.22, 4.23 and 4.24 it is observed that the simulated flow results closely match the measured flow results with coefficients of determination (R^2) of 0.9996, 0.999 and

0.9997 respectively. Higher values of R^2 indicate that less error variance and typically values greater than 0.5 are considered acceptable (Santhi *et al.*, 2001).

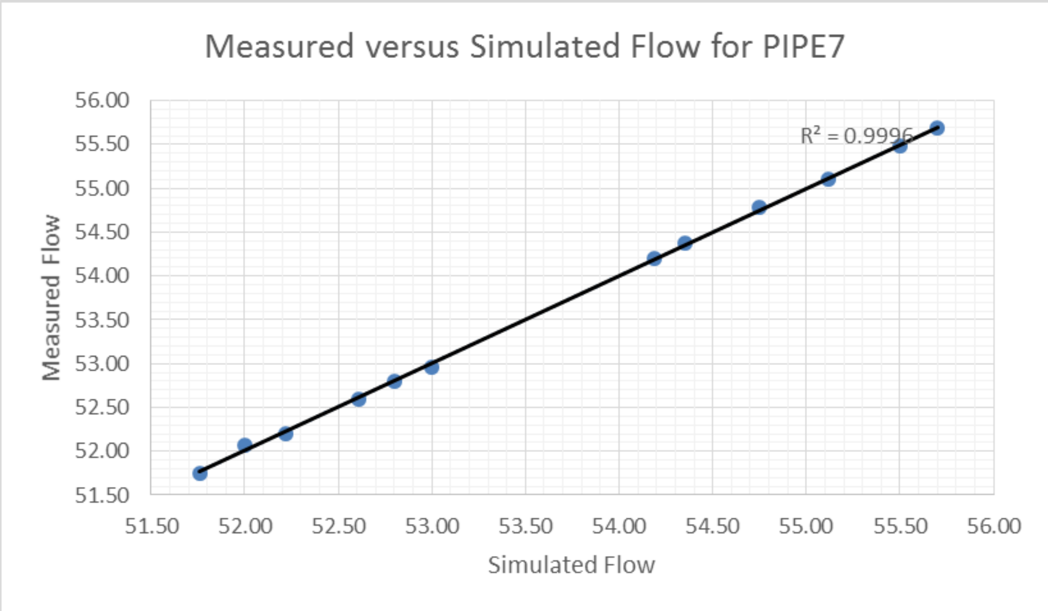


Figure 4.22: Measured versus simulated flow for Pipe 7 PCEA zone

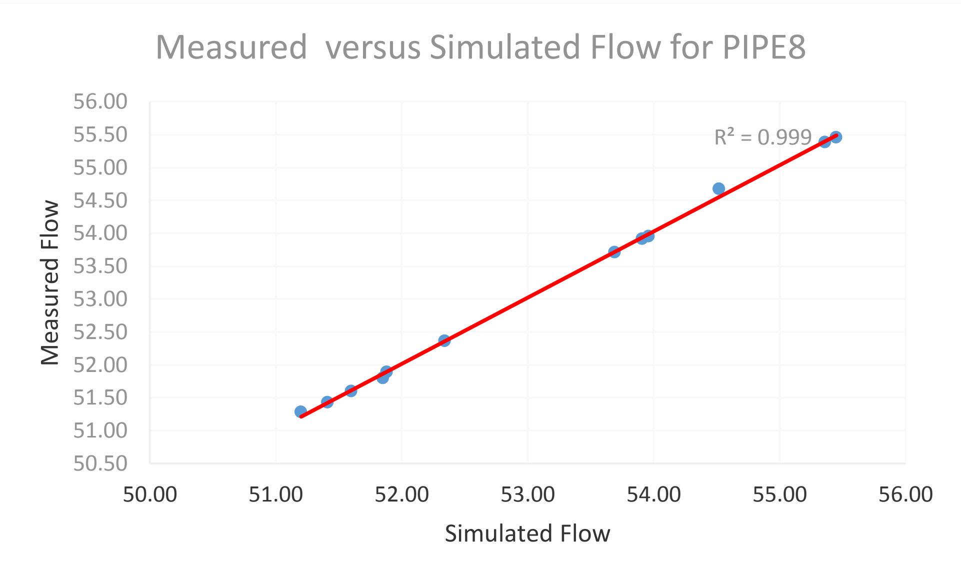


Figure 4.23: Measured versus simulated for Pipe 8 in PCEA zone

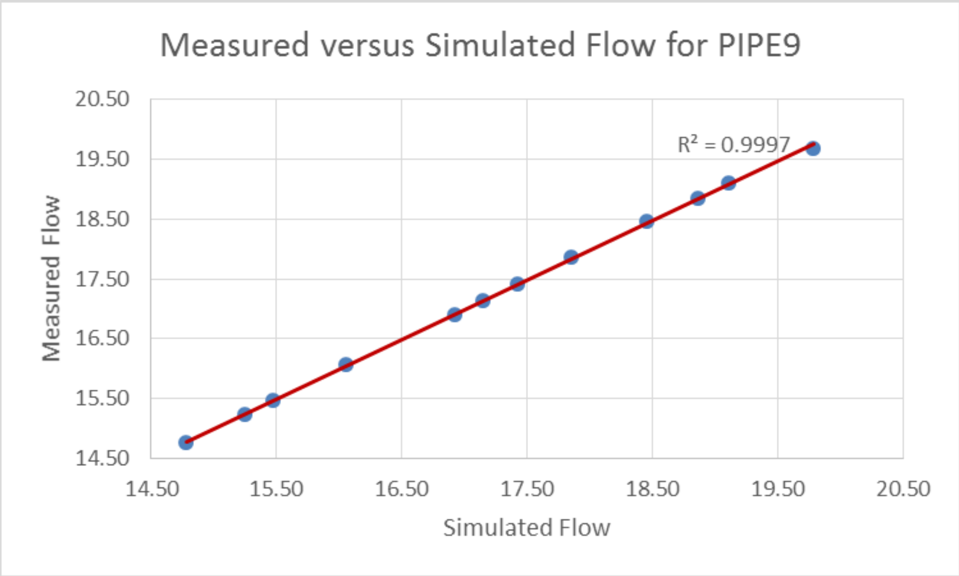


Figure 4.24: Measured versus simulated flow for Pipe 9 in PCEA zone

From Figures 4.25, 4.26 and 4.27 it is observed that the simulated headloss results closely match the calculated headloss results with coefficients of determination (R^2) of 0.9694, 0.9414, 0.9581 respectively. This indicates that the model performance is satisfactory. Higher values of R^2 indicate less error variance and typically values greater than 0.5 are considered acceptable (Santhi *et al.*, 2001).

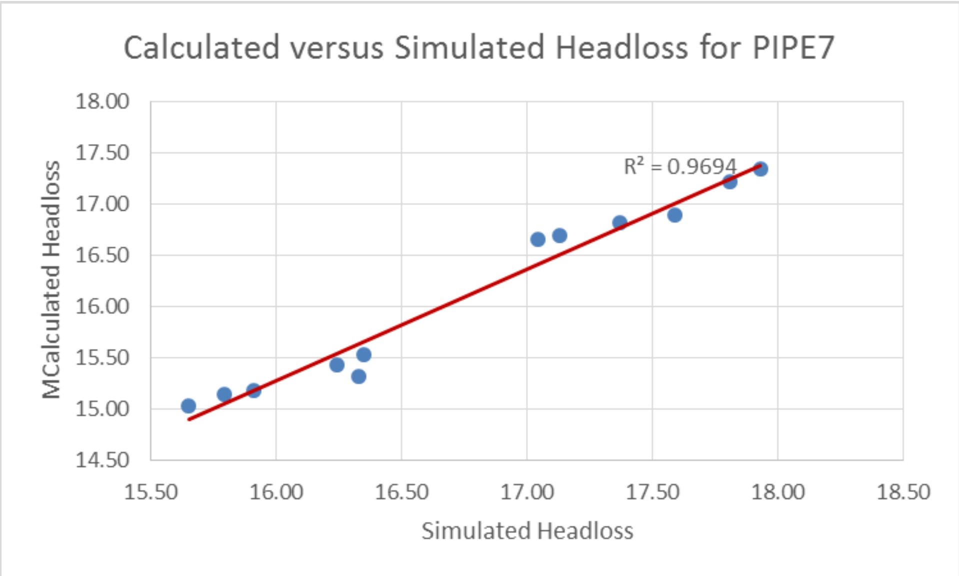


Figure 4.25: Calculated versus simulated headloss for Pipe 7 in PCEA zone

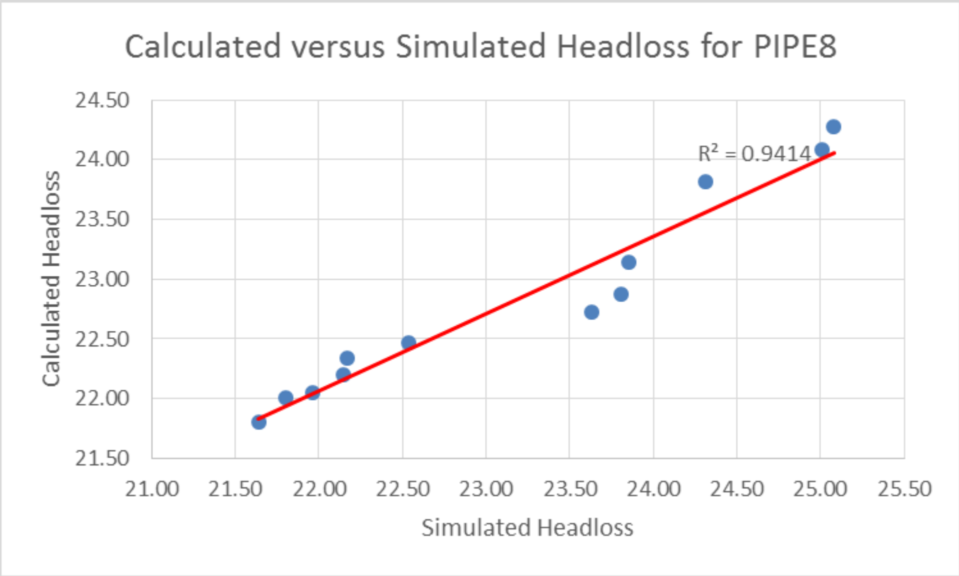


Figure 4.26: Calculated versus simulated headloss for Pipe 8 in PCEA zone

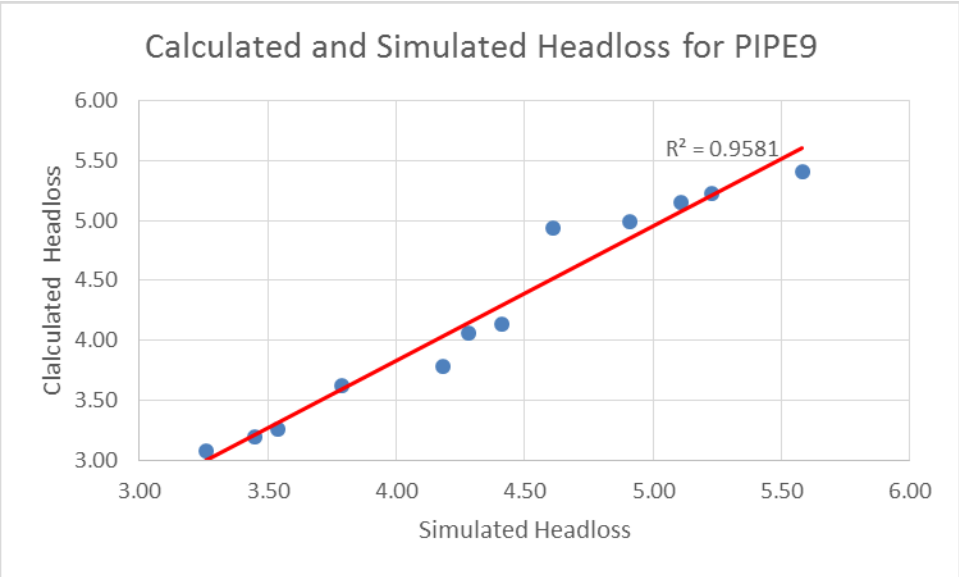


Figure 4.27: Calculated versus simulated headloss for Pipe 9 in PCEA zone

4.2.7 Measured versus simulated flow and headloss for Town 1 zone pipes

From Figures 4.28, 4.29 and 4.30 it is observed that the simulated results closely match the measured results with a coefficient of determination (R^2) of 0.9985, 0.9978 and 0.9975 respectively. The high coefficient of determination confirms that the model efficiency is also high.

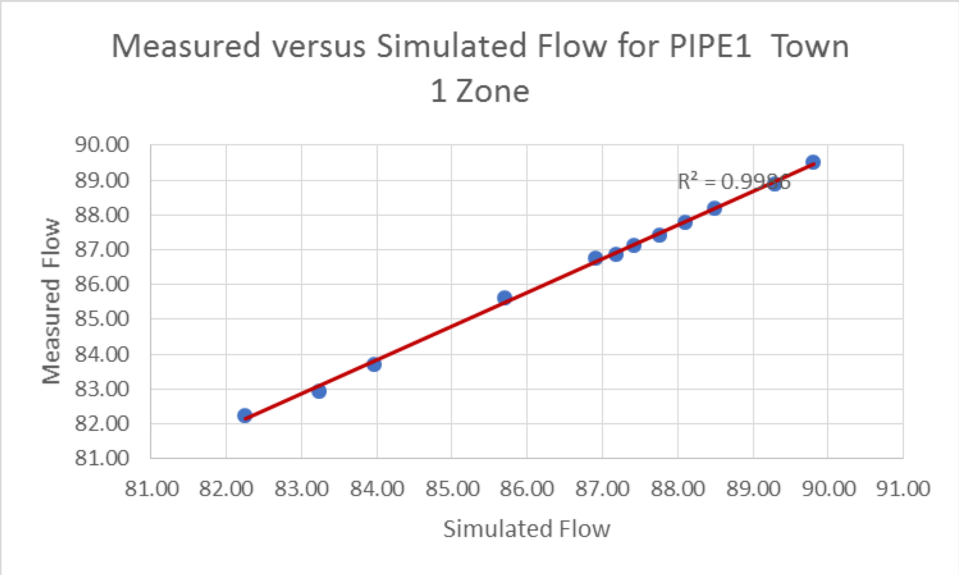


Figure 4.28: Measured Vs. Simulated flow for Pipe 1 in Town 1 zone

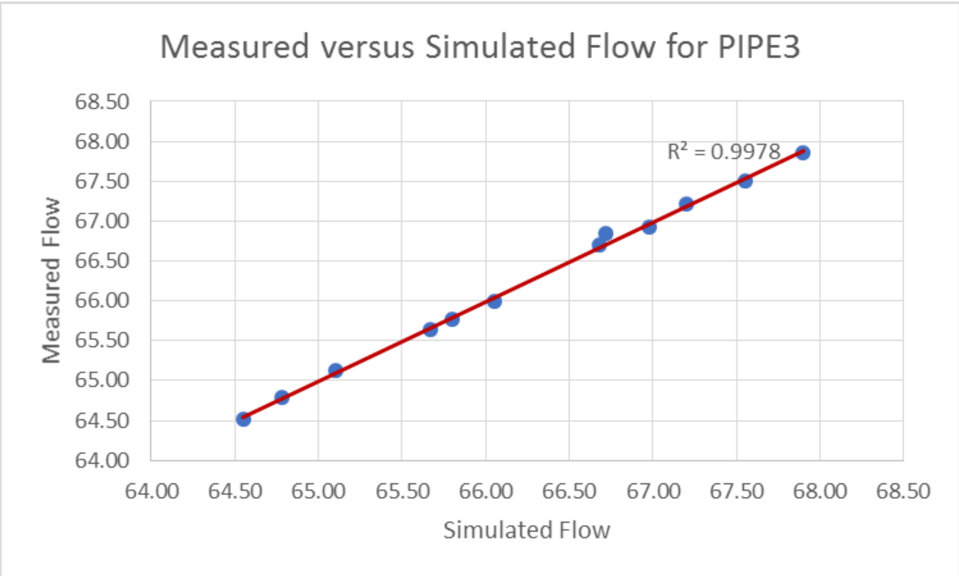
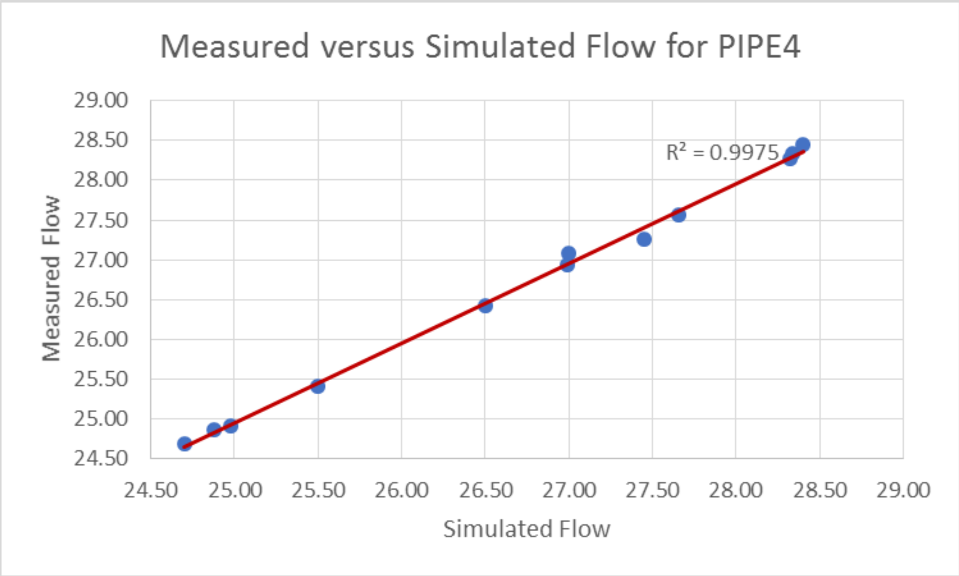


Figure 4.29: Measured Vs. Simulated flow for Pipe 3 in Town 1 zone



Fiureg 4.30: Measured Vs. Simulated flow for Pipe 4 in Town 1 zone

From Figures 4.31, 4.32 and 4.33 it is observed that the simulated results closely match the measured results with coefficients of determination (R^2) of 0.9786, 0.9466 and 0.9951 respectively. This indicates that the model performance is satisfactory. Higher values of R^2 indicate that less error variance and typically values greater than 0.5 are considered acceptable (Santhi *et al.*, 2001).

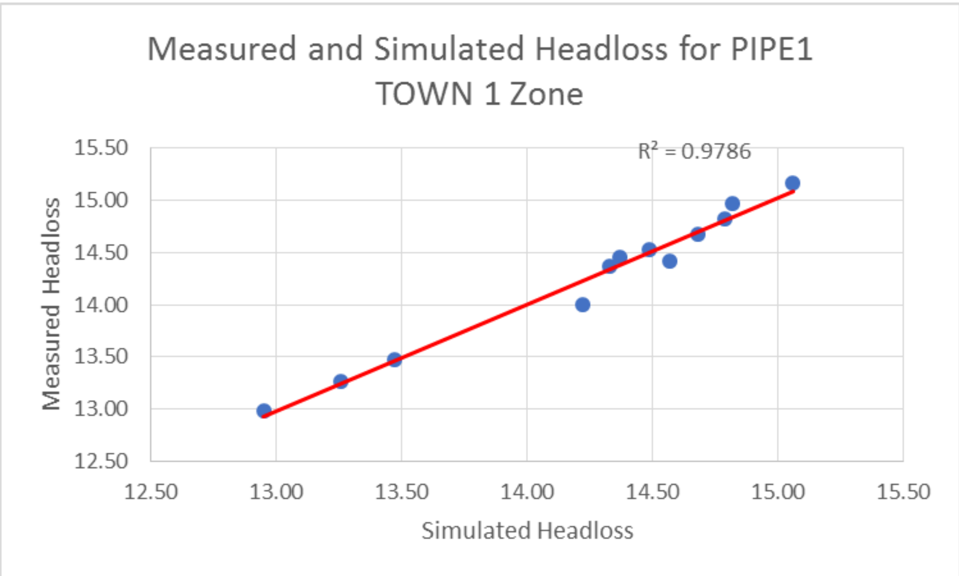


Figure 4.31: Calculated Vs. Simulated headloss for Pipe 1 in Town 1 zone

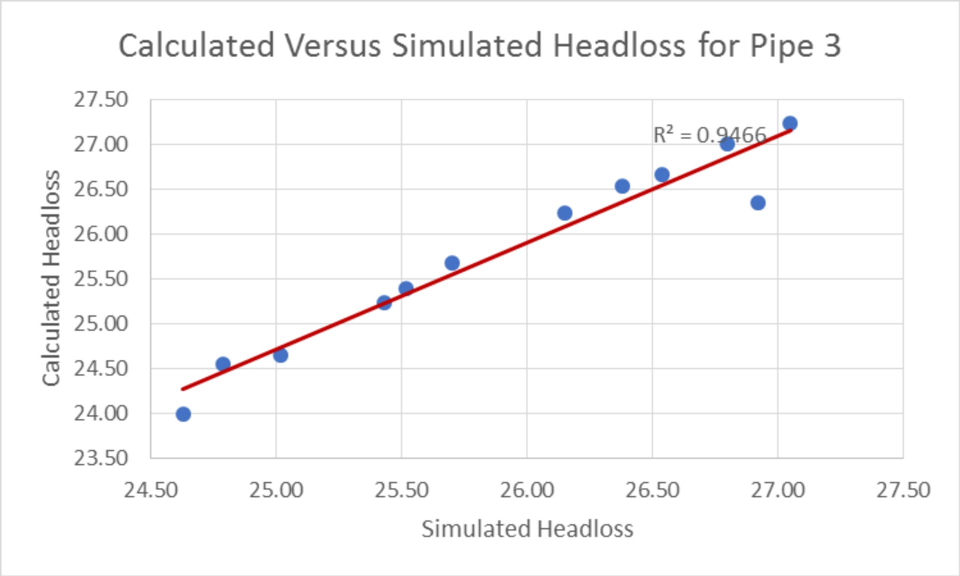


Figure 4.32: Measured Vs. Simulated headloss for Pipe 3 in Town 1 zone

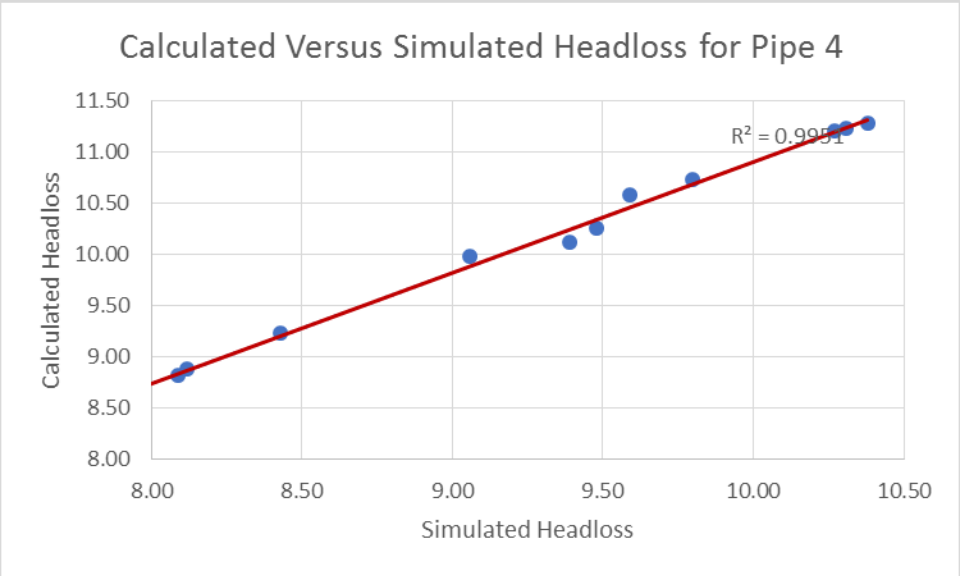


Figure 4.33: Calculated Vs. Simulated headloss for Pipe 4 in Town 1 zone

4.2.8 Measured Versus Simulated Flow and Headloss for Town 2 Zone Pipes

From Figures 4.34, 4.35 and 4.36 it is observed that the simulated results closely match the measured results with coefficients of determination (R^2) of 0.9995, 1 and 0.9986 respectively. This indicates that the model performance is satisfactory. Higher values of R^2 indicate that less error variance and typically values greater than 0.5 are considered acceptable (Santhi *et al.*, 2001).

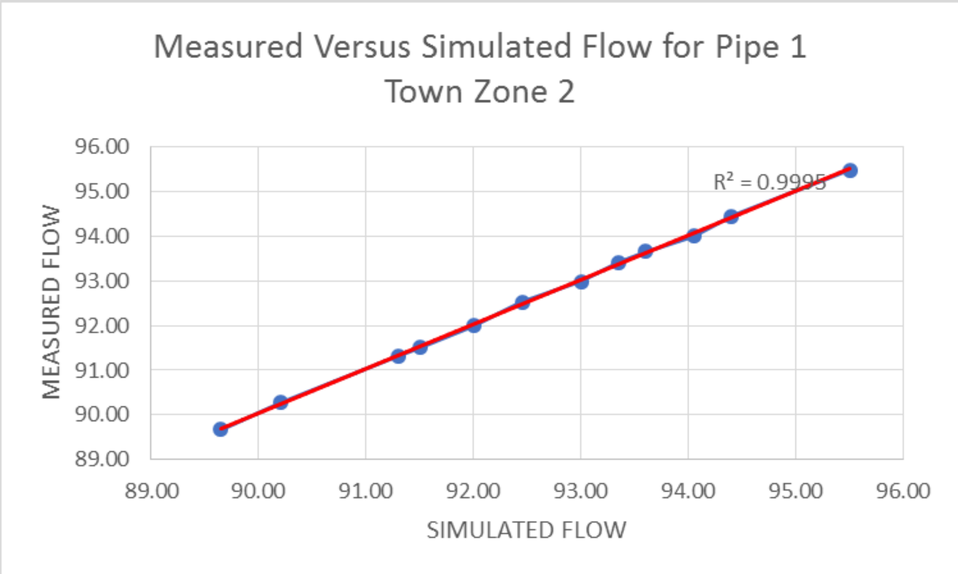


Figure 4.34: Measured Vs. Simulated flow for Pipe 1 in Town 2 zone

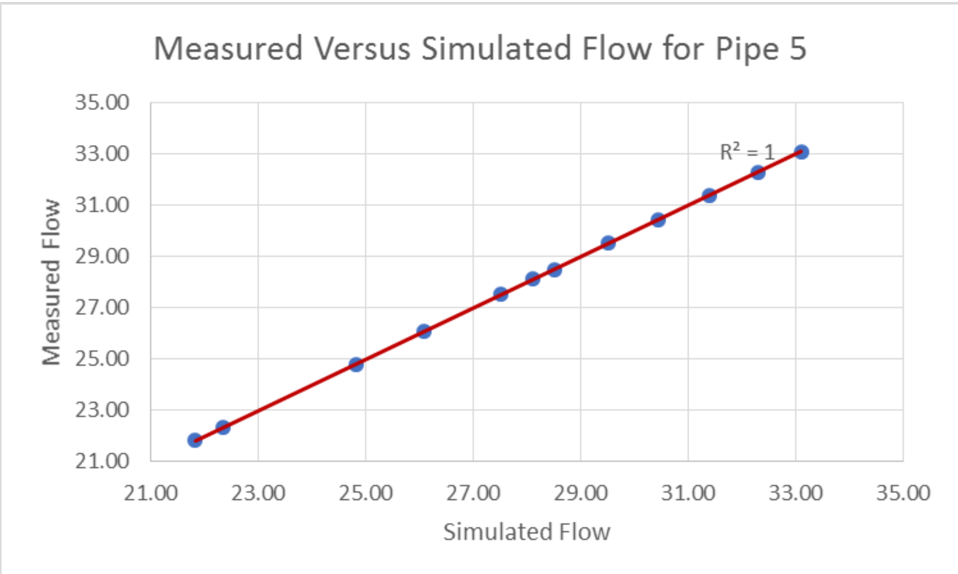


Figure 4.35: Measured Vs. Simulated flow for Pipe 5 in Town 2 zone

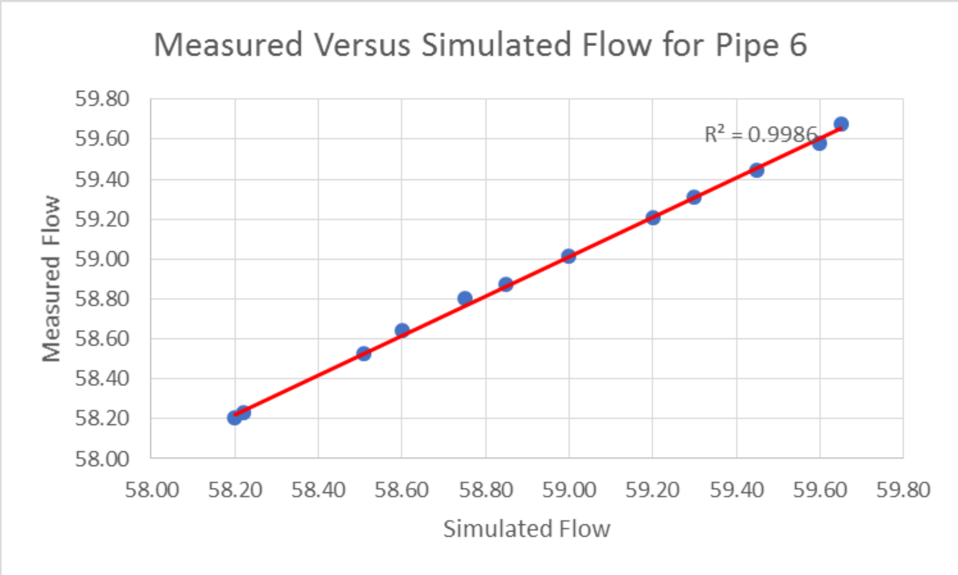


Figure 4.36: Measured Vs. Simulated flow for Pipe 6 in Town 2 zone

From Figures 4.37, 4.38 and 4.39 it is observed that the simulated results closely match the measured results with coefficients of determination ($R^{2)}$ of 0.9995, 0.9797, 0.9466 respectively. This indicates that the model performance is satisfactory. Higher values of R^2 indicate that less error variance and typically values greater than 0.5 are considered acceptable (Santhi *et al.*, 2001).

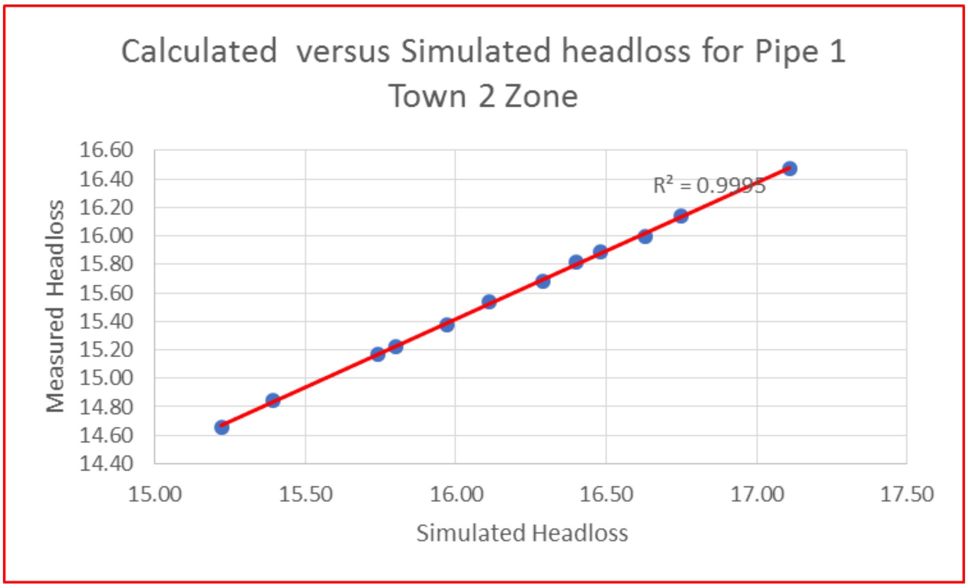


Figure 4.37: Calculated Vs. Simulated headloss for Pipe 1 in Town 2 zone

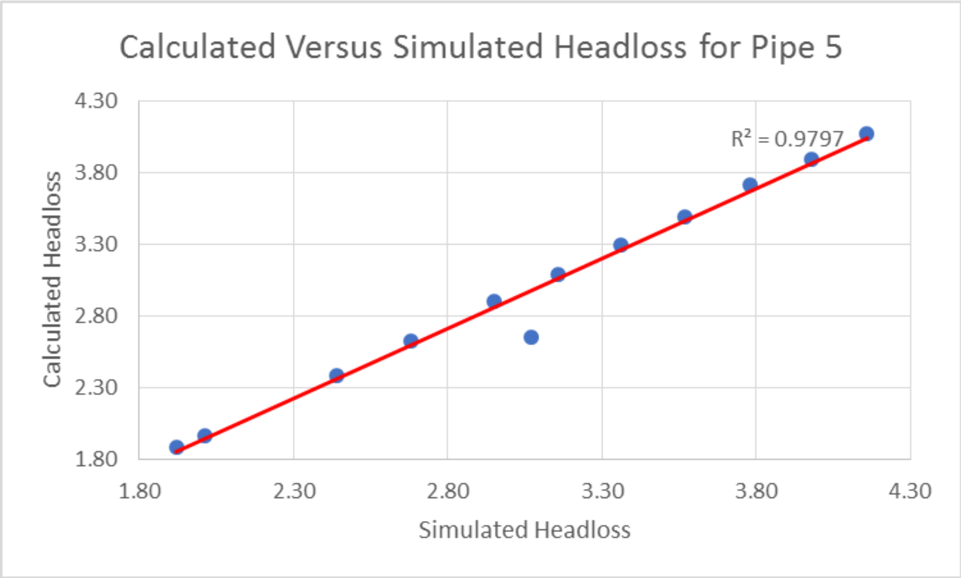


Figure 4.38: Calculated Vs. Simulated headloss for Pipe 5 in Town 2 zone

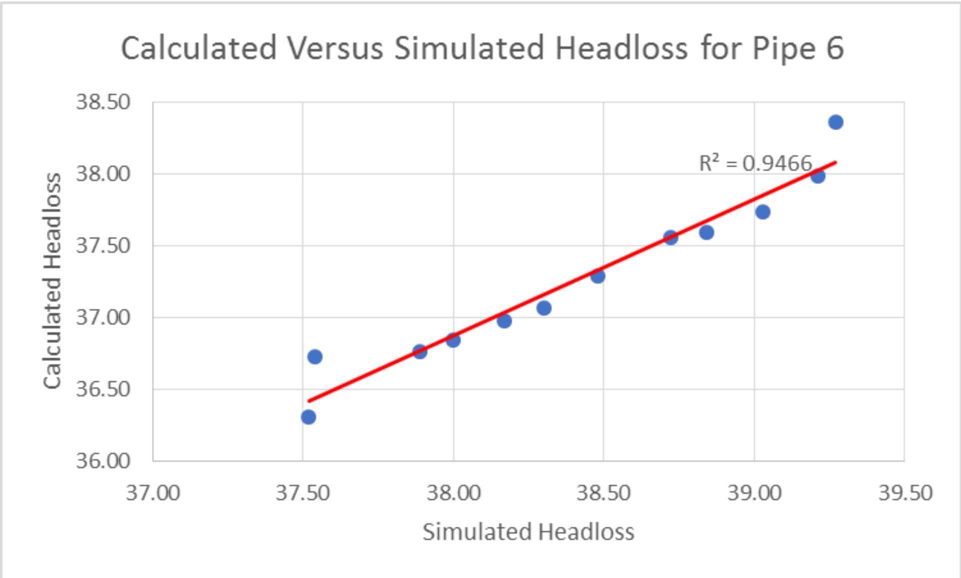


Figure 4.39: Calculated Vs. Simulated headloss for Pipe 6 in Town 2 zone

4.2.9 Measured versus Simulated Chlorine Levels for Pipe 9

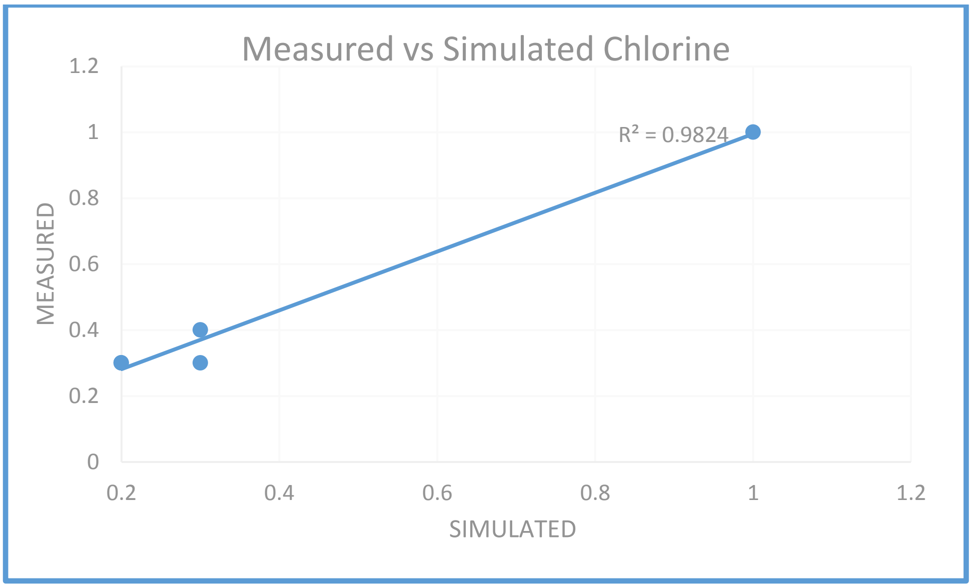


Figure 4.40: Measured Vs. Simulated Chlorine levels for Pipe 9

From Figure 4.40 it is observed that the simulated results closely match the measured results with a coefficient of determination (R^2) of 0.9824. The high coefficient of determination confirms that the model efficiency is also high.

Table 4.26: Model Performance Based on Different Statistical Criteria

Statistical Criterion	Performance Level	Optimal Value/Range
Nash-Sutcliffe Efficiency Coefficient (N-S)	0.999999	-∞ - 1
Sum of Squares Error (SSE)	0.01	0
Percent Bias(PB)	0.05	0
Root Mean Square Error (RMSE)	0.03	0

From Table 4.26, it is observed that the model performed well based on results of Nash-Sutcliffe Efficiency, Sum of Squares Error, Percent Bias, and Root Mean Square Error since the calibration results are within the acceptable ranges.

4.2.10 Model Sensitivity

To establish the changes to the output dependent variables, independent variables were varied. The identified independent variables that were varied were; pipe roughness, pipe diameter, pipe length and initial chlorine concentrations. Each of the variable was reduced and also increased by 10% for the entire network and the changes to the output variables were presented in a graphical format as summarized in Figure 4.41.

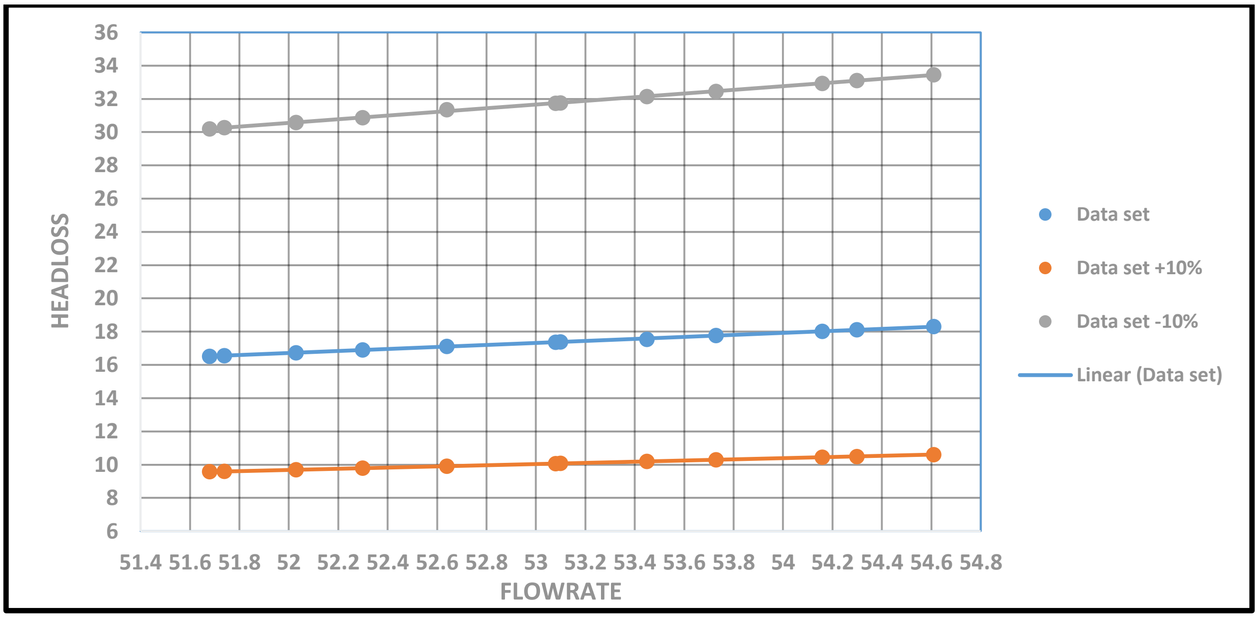


Figure 4.41: Effect of Varied Independent Variables on Headloss for PIPE 1 County Zone

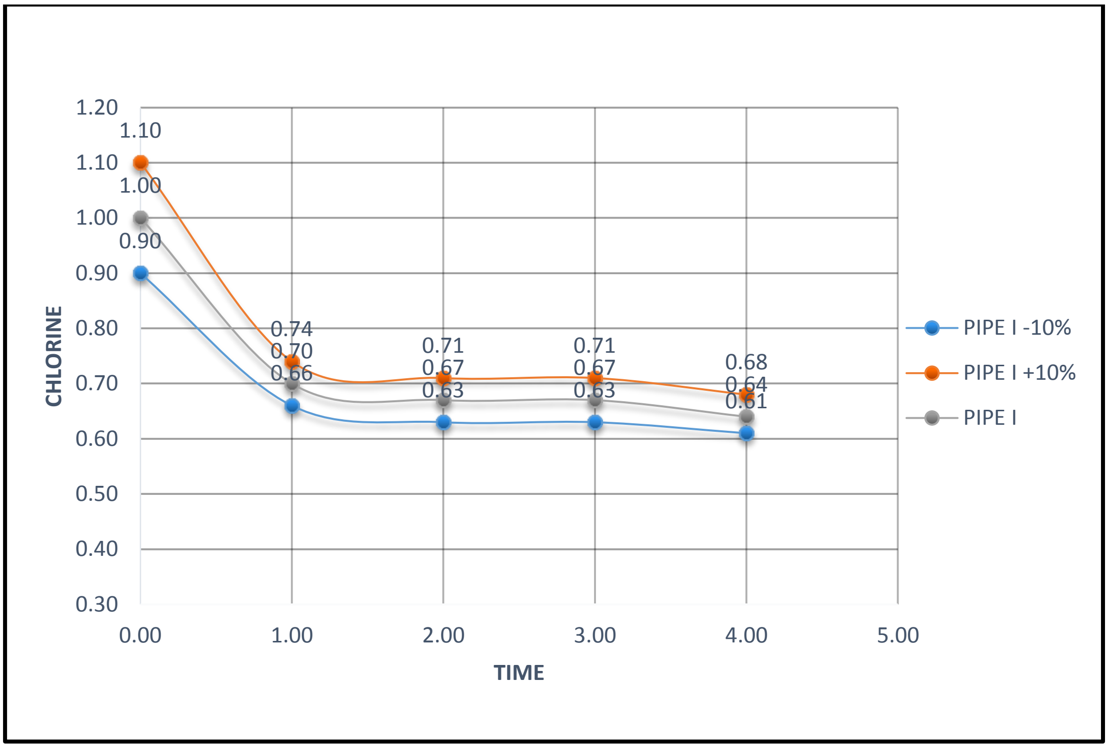


Figure 4.42: Effect of Varied Independent Variables on Chlorine Decay

Figure 4.41 shows the head loss output values which were obtained with the +10% and -10% variation of pipe diameter, roughness factor and pipe length being varied all at the same time. However, Variation of diameter by +10% and -10% while holding other variables constant resulted to lower and higher headloss values respectively. Similarly, variation of roughness factor and pipe length by +10% resulted to higher headloss values while variation of both by -10% each resulted to lower headloss values.

From figure 4.42, the results indicate that the initial chlorine concentration (1.0mg/l) was varied by +10%(1.1mg/l) and -10%(0.9mg/l). A plot of the respective chlorine levels over time indicates a well sensitive model.

However, variation of diameter produced significant variation of headloss as compared to roughness, length and initial chlorine concentration. This means pipe diameter is the most sensitive independent variable to headloss.

4.3 Impact of Chlorine Decay on the System Hydraulic Performance

4.3.1 Determination of Bulk Reaction Rate K_b

Table 4.27 presents results for measured residual chlorine concentration, (C_t/C_o) and natural log of (C_t/C_o) where C_t was concentration at time t and C_o was concentration at time zero.

Table 4.27: Measured Residual Chlorine Over Time as a Ratio of Initial Concentration

TIME (Hr)	Measured Residual Chlorine(mg/l)	C_t/C_0	Ln (Ct/C_0)
0	1	1	0
0.2	0.4	0.4	-0.9162907
0.5	0.3	0.3	-1.2039728
1	0.3	0.3	-1.2039728
1.5	0.3	0.3	-1.2039728
2	0.3	0.3	-1.2039728
3	0.2	0.2	-1.6094379
6	0.2	0.2	-1.6094379

K_b which was the slope of the line of a plot of Ln (Ct/C_0) against time equals to -1.

4.3.2 Spatial and Temporal Chlorine Levels

Figure 4.43 presents results for 1.00hrs chlorine levels for PIPE0, PIPE1 and PIPE2 are 0.7 mg/l, 0.42mg/l and 0.36 mg/l mg/l respectively. Similarly, the results for the TANK, NODE2, NODE3 and NODE4 are 0.96 mg/l, 0.46 mg/l, 0.39 mg/l and 0.33 mg/l respectively. The diameters for PIPE0, PIPE1 and PIPE2 are 200mm, 150mm and 100mm respectively. The results indicate chlorine decay over time along a pipe is dependent on the diameter of the pipe.

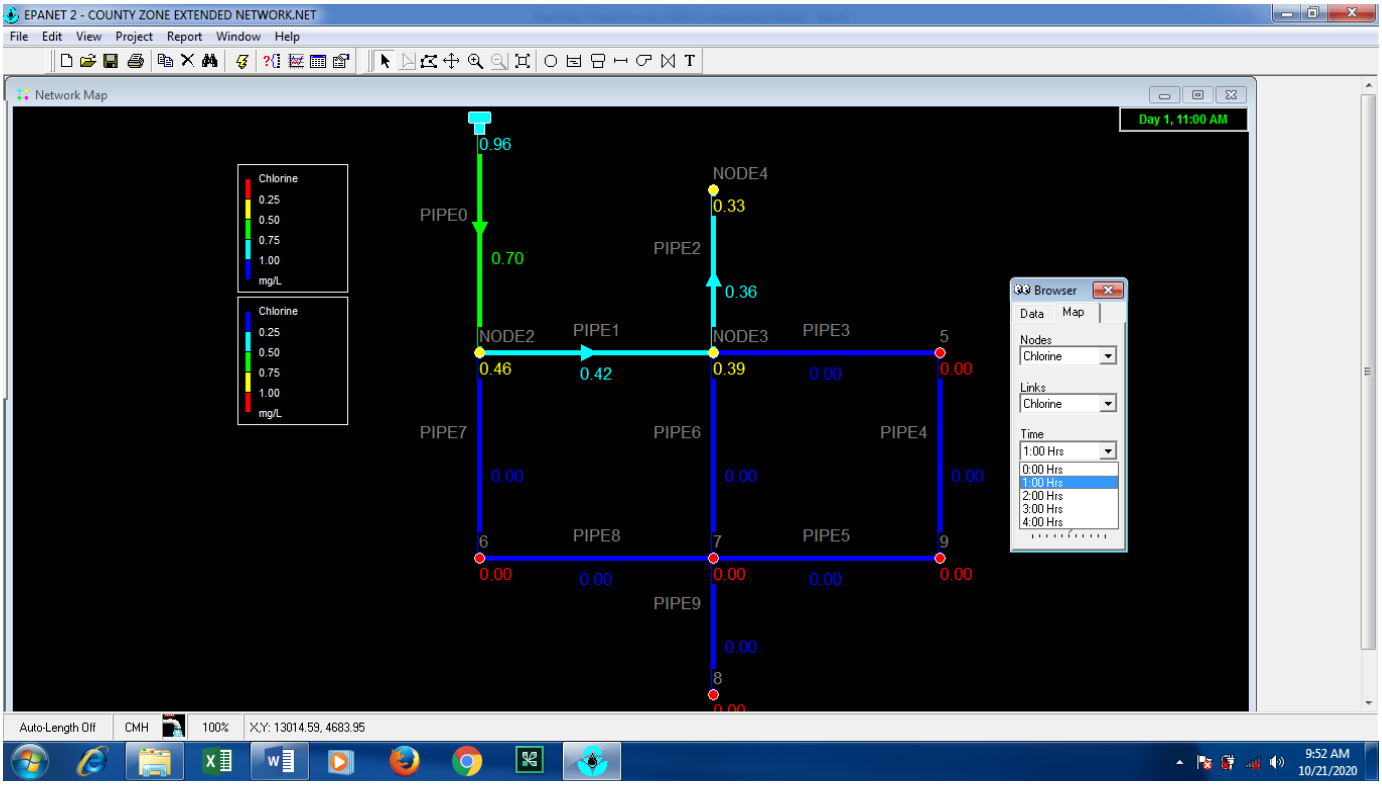


Figure 4.43: 1.00 Hrs Chlorine Levels in Pipes and Nodes

4.3.3 Chlorine Decay with Respect to Pipe Diameter

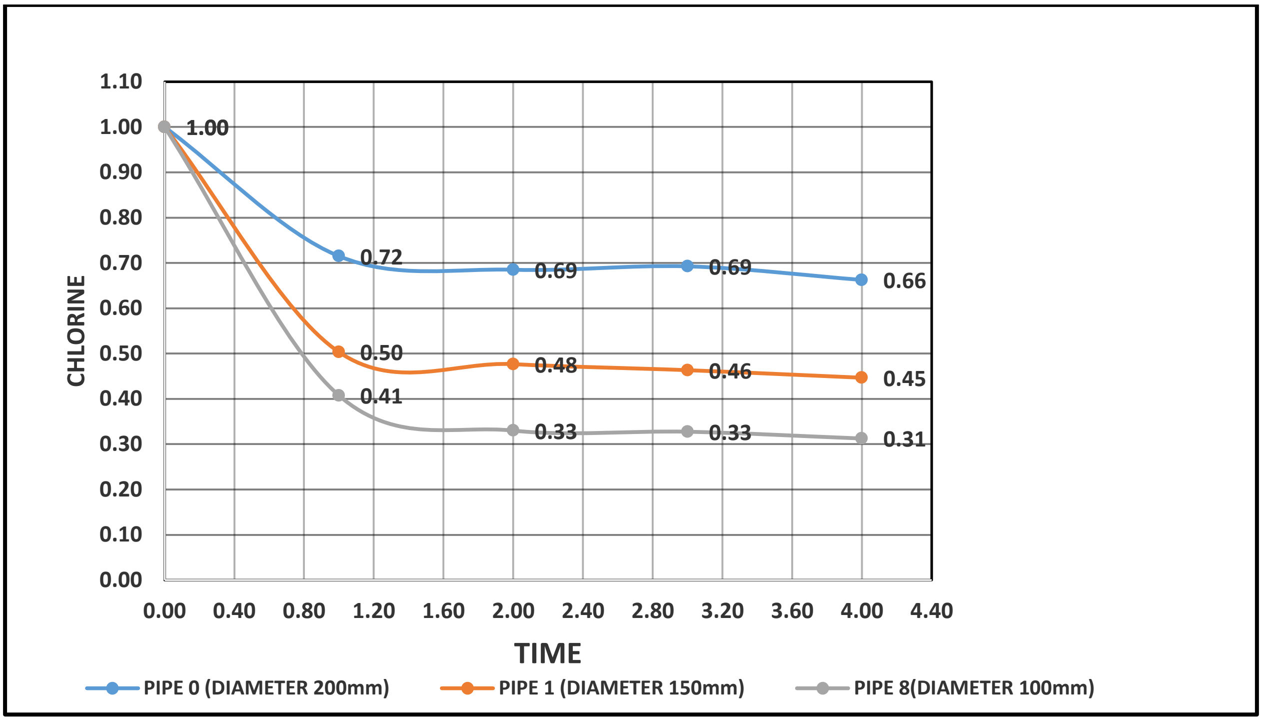


Figure 4.44: Chlorine Levels vs. Time for Different Pipe Diameters

A graphical presentation of residual chlorine over time with respect to pipe diameter is depicted in Figure 4.44. The results show that the initial chlorine concentration is 1.00 mg/l which decays over time with respect to pipe diameter. Chlorine concentration in mg/l for the 200mm diameter pipe was 0.72, 0.69, 0.69 and 0.66 after 1hr, 2 hrs, 3 hrs and 4 hrs respectively. However, for a 100 mm diameter pipe it is observed that the results are 0.41, 0.33, 0.33, and 0.31 mg/l after 1hr, 2 hrs, 3 hrs and 4 hrs respectively. The above results indicate that chlorine decay in smaller diameter pipes faster than in a larger diameter pipes. This is because greater diameter means smaller surface-to-volume ratio and subsequently a slower chlorine decay rate and vice versa. In addition, the smaller the pipe the higher the friction along the pipe leading to more chlorine decay.

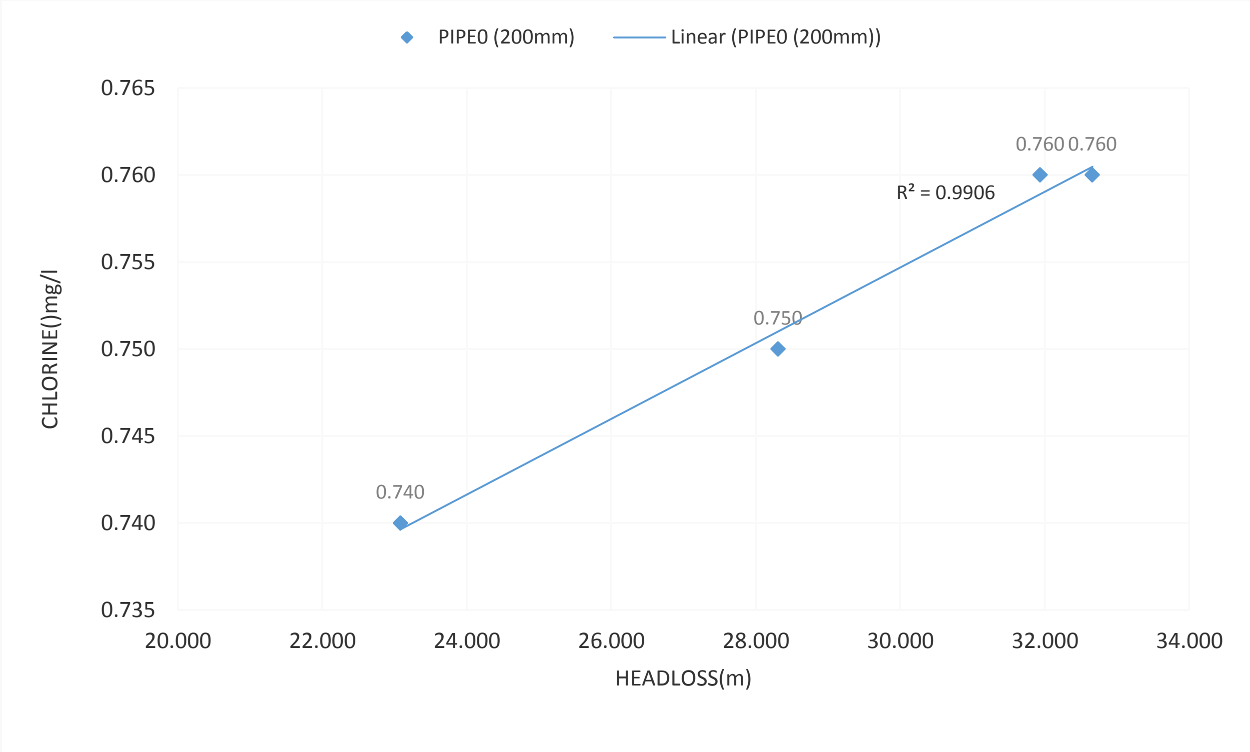


Figure 4.45: Average Residual Chlorine Versus Average Headloss for a 200mm Diameter Pipe

From Figure 4.45, the results show a linear relationship of residual chlorine and headloss is presented with a coefficient of correlation of 0.9906. This indicates that as chlorine decays over time biofilm accumulates along pipe walls that then causes internal pipe roughness. Increased pipe roughness contributes significantly to increase in pipe headloss. Higher pipe headloss indicates reduced pressures and therefore higher flows are required to sustain both base and peak water demand.

Minimum chlorine level for drinking water is 0.2mg/l and the results in Figure 4.43 indicate that chlorine decay in a smaller diameter pipe is faster than in a larger diameter pipe. Lower chlorine levels below the minimum standard affects water quality. The results in Figures 4.43 and 4.44 show that chlorine decay with respect to pipe diameter has a significant impact on both hydraulic performance and water quality of a water distribution network.

From Table 4.28 temporal chlorine levels for County, PCEA Town 1 and Town 2 zones were presented with respect to 200mm, 150mm and 100mm pipe diameters. Average residual chlorine levels were also presented for each zone with respect to pipe three pipe diameters indicated above. From the results presented in Table 4.28 initial chlorine concentrations were 1.00mg/l for all zones and respective chlorine levels for 1.00hrs, 2.00hrs, 3.00hrs and 4.00hrs decayed over time with respect to pipe diameter.

Table 4.28: Predicted Chlorine Levels in mg/l for Maralal Water Distribution Network

PIPE DIAMETER	TIME (hrs)	COUNTY	PCEA	TOWN 1	TOWN 2	AVERAGE RESIDUAL CHLORINE(mg/l)
200mm	0.00	1.00	1.00	1.00	1.00	1.00
	1.00	0.70	0.70	0.73	0.73	0.72
	2.00	0.67	0.67	0.70	0.70	0.69
	3.00	0.70	0.67	0.70	0.70	0.69
	4.00	0.67	0.64	0.67	0.67	0.66
150mm	0.00	1.00		1.00	1.00	1.00
	1.00	0.42		0.46	0.46	0.50
	2.00	0.39		0.44	0.44	0.48
	3.00	0.47		0.47	0.47	0.46
	4.00	0.45		0.45	0.45	0.45
100mm	0.00	1.00	1.00	1.00	1.00	1.00
	1.00	0.36	0.45	0.37	0.39	0.41
	2.00	0.28	0.43	0.34	0.37	0.33
	3.00	0.37	0.45	0.38	0.40	0.33
	4.00	0.35	0.43	0.36	0.38	0.31

Tables 4.29, 4.30 and 4.31indicated the variation of headloss and chlorine decay for County, PCEA, Town 1 and Town 2 zones with respect to 200mm, 150mm and 100mm pipe diameters. From table 4.29 variation of headloss and chlorine decay with respect to 200mm and 150mm pipe diameters were presented.

Table 4.29: Variation of Headloss and Chlorine Decay with Respect to 200mm and 150mm Pipe Diameter

DIAMETE R (mm)	TIME (hrs)	COUNTY HEADLOSS (m)	RESIDUAL CHLORINE (mg/l)	PCEA HEADLOSS (m)	RESIDUAL CHLORINE (mg/l)	TOWN 1 HEADLOSS (m)	RESIDUAL CHLORINE (mg/l)	TOWN 2 HEADLOSS (m)	RESIDUAL CHLORINE(mg/l)
200	0.00	19.56	1.00	19.56	1.00	27.68	1.00	27.06	1.00
	1.00	19.56	0.70	19.56	0.70	27.68	0.73	27.06	0.73
	2.00	41.43	0.67	28.36	0.67	40.13	0.70	39.25	0.70
	3.00	41.43	0.70	28.36	0.67	40.13	0.70	39.25	0.70
	4.00	19.56	0.67	19.56	0.64	27.68	0.67	27.06	0.67
150	0.00	7.32	1.00			13.03	1.00	14.63	1.00
	1.00	7.32	0.42			13.03	0.46	14.63	0.46
	2.00	15.52	0.39			18.89	0.44	21.22	0.44
	3.00	15.52	0.46			18.89	0.47	21.22	0.47
	4.00	7.32	0.45			13.03	0.45	14.63	0.45

Table 4.30: Variation of Headloss and Chlorine Decay with Respect to 100mm Pipe Diameter (COUNTY and PCEA Zones)

TIME (hrs)	PIPE 2 HEADLOSS (m)	RESIDUAL CHLORINE (mg/l)	PIPE 7 HEADLOSS (m)	RESIDUAL CHLORINE (mg/l)	PIPE 8 HEADLOSS (m)	RESIDUAL CHLORINE (mg/l)	PIPE 9 HEADLOSS (m)	RESIDUAL CHLORINE(mg/l)
0.00	1.14	1.00	16.35	1.00	16.10	1.00	15.40	1.00
1.00	1.14	0.36	16.35	0.46	16.10	0.38	15.40	0.38
2.00	2.42	0.28	23.71	0.44	23.35	0.35	22.34	0.27
3.00	2.42	0.37	23.71	0.46	23.35	0.38	22.34	0.29
4.00	1.14	0.35	16.35	0.44	16.10	0.36	15.40	0.28

Table 4.31: Variation of Headloss and Chlorine Decay with Respect to 100mm Pipe Diameter (TOWN 1 and TOWN 2 Zones)

	PIPE 3		PIPE 4		PIPE 5		PIPE 6	
TIME (hrs)	HEADLOSS (m)	RESIDUAL CHLORINE (mg/l)	HEAD LOSS (m)	RESIDUAL CHLORINE (mg/l)	HEADLOSS (m)	RESIDUAL CHLORINE (mg/l)	HEADLOSS (m)	RESIDUAL CHLORINE(mg/l)
0.00	10.45	1.00	6.88	1.00	3.38	1.00	13.24	1.00
1.00	10.45	0.39	6.88	0.43	3.38	0.31	13.24	0.37
2.00	15.15	0.37	9.97	0.37	4.90	0.27	19.19	0.34
3.00	15.15	0.40	9.97	0.41	4.90	0.31	19.19	0.38
4.00	10.45	0.38	6.88	0.40	3.38	0.29	13.24	0.36

From Figure 46 predicted Chlorine decay over a period of four hours for PIPE0, PIPE1 and PIPE2 in County zone were 0.7mg/l, 0.67mg/l, 0.7mg/l, 0.67mg/l and 0.42mg/l, 0.39mg/l, 0.47gm/l, 0.45gm/l and 0.36mg/l, 0.28mg/l, 0.37mg/l, 0.35mg/l respectively.

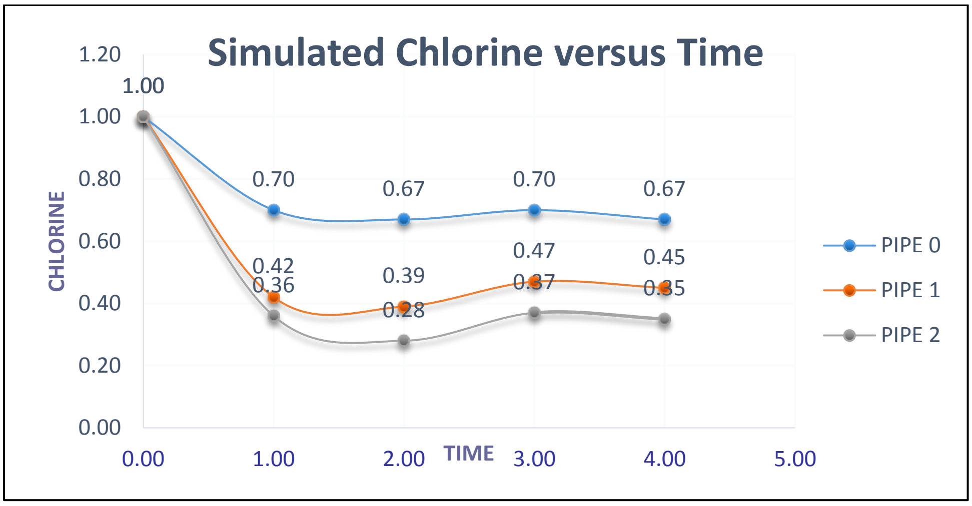


Figure 4.46: County Zone Chlorine Decay over Time

From Figure 47 simulated Chlorine decay over a period of four hours for PIPE0, PIPE7, PIPE8 and PIPE9 in PCEA zone were 0.7mg/l, 0.67mg/l, 0.67mg/l, 0.64mg/l and 0.46mg/l, 0.44mg/l, 0.46gm/l, 0.44gm/l and 0.38mg/l, 0.35mg/l, 0.38mg/l, 0.36mg/l and 0.38mg/l, 0.27mg/l, 0.29mg/l, 0.28mg/l respectively.

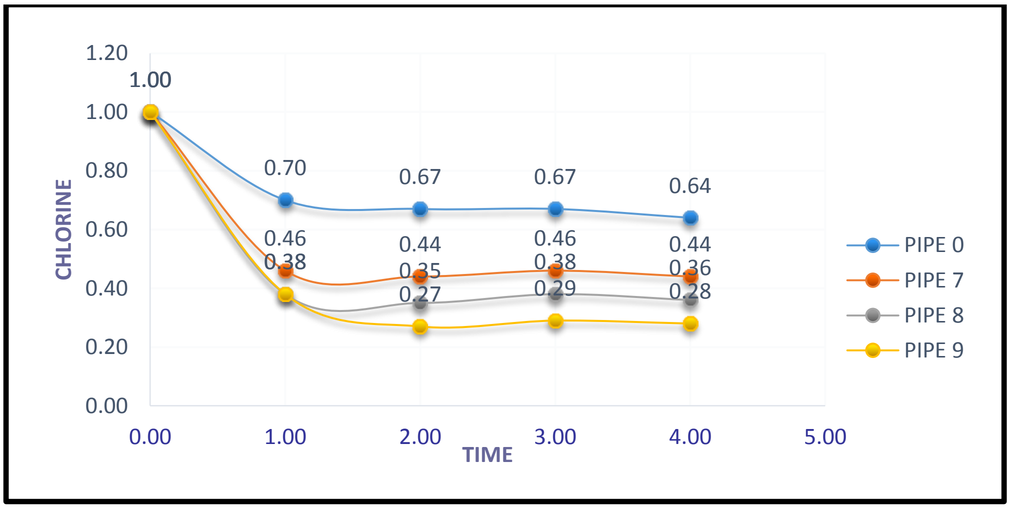


Figure 4.47: PCEA Zone Chlorine Decay over Time

From Figure 48 simulated Chlorine decay over a period of four hours for PIPE0, PIPE1, PIPE3 and PIPE4 in Town zone 1 were 0.73mg/l, 0.7mg/l, 0.7mg/l, 0.74mg/l and 0.46mg/l, 0.44mg/l, 0.47gm/l, 0.45gm/l and 0.43mg/l, 0.37mg/l, 0.41mg/l, 0.4mg/l and 0.39mg/l, 0.37mg/l, 0.40mg/l, 0.38mg/l respectively.

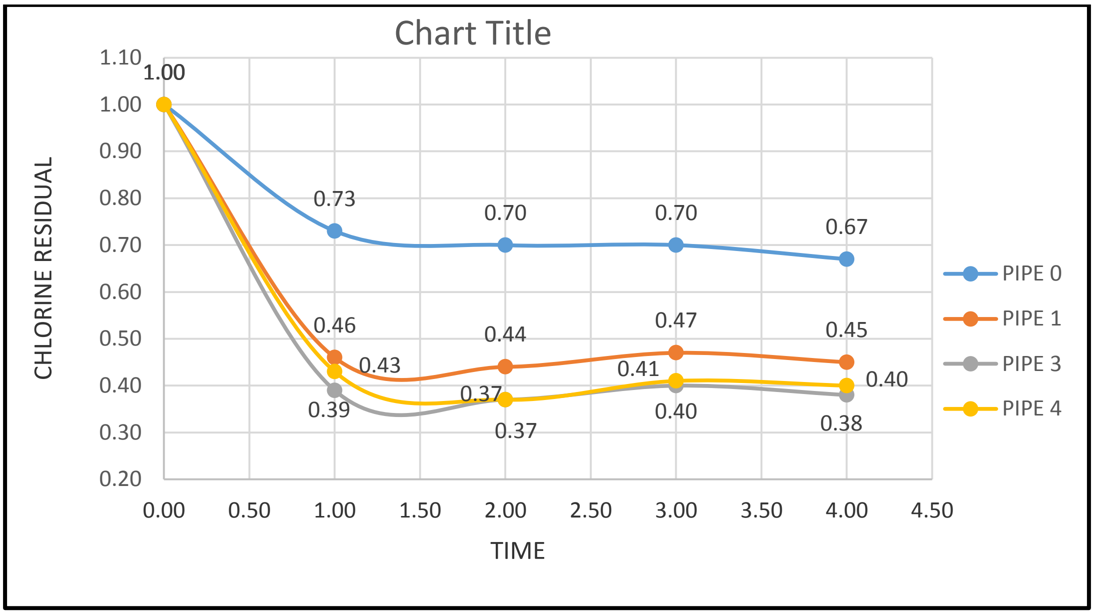


Figure 4.48: Town Zone 1 Chlorine Decay over Time

From Figure 49 predicted Chlorine decay over a period of four hours for PIPE0, PIPE1, PIPE6 and PIPE5 in Town zone 2 were 0.73mg/l, 0.7mg/l, 0.7mg/l, 0.67mg/l and 0.46mg/l, 0.44mg/l, 0.47gm/l, 0.45gm/l and 0.37mg/l, 0.34mg/l, 0.38mg/l, 0.36mg/l and 0.31mg/l, 0.27mg/l, 0.31mg/l, 0.29mg/l respectively.

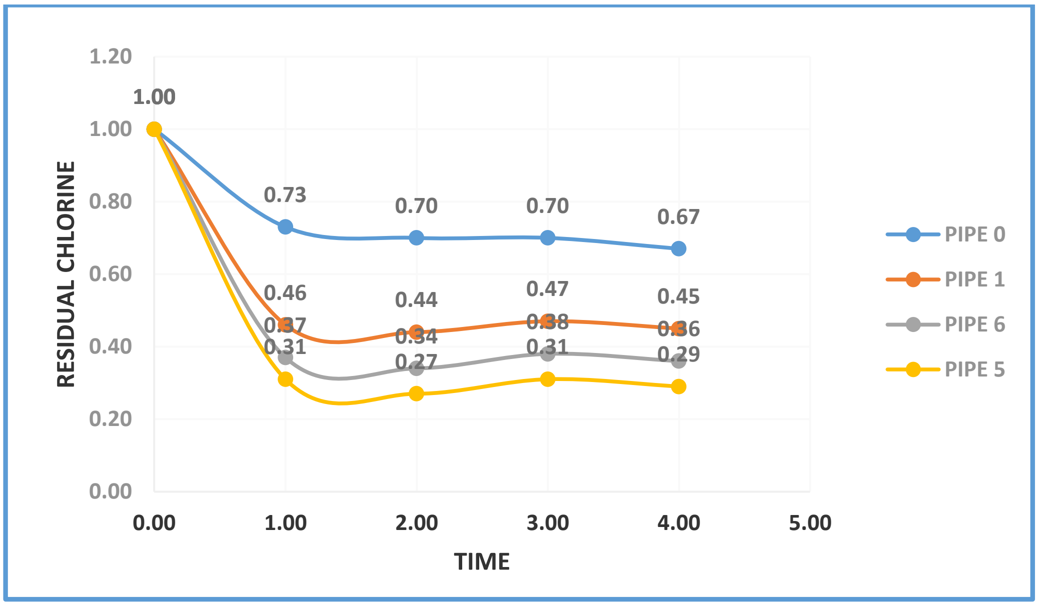


Figure 4.49: Town Zone 2 Chlorine Decay over Time

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

From results and inferences of this study, it could be concluded that:

i. A regression analysis for pipe headloss as a function of flowrate was successfully applied to determine the current values for roughness factor of each pipe for the Maralal water distribution network;

ii. EPANET model was successfully used as a tool to efficiently simulate and predict at any time the hydraulic performance and water quality for Maralal water distribution system; and

iii. Chlorine decay significantly affected both the hydraulic performance and water quality for the Maralal water distribution network in that, the chlorine decayed faster in pipes with smaller diameters compared to those with larger diameters. In addition, the residual chlorine levels were lower than the recommended 0.2mg/l thus compromises the water quality delivered to consumers served by the Maralal water distribution system.

5.2 Recommendations

Based on the results of this study, further research work could be conducted to:

i. Determine roughness factors of pipes in a water distribution network using flow and headloss data collected over a longer period of time for a continuous flow system,

ii. Carry out simulations in a larger network that consists of many loops. For instance, the solution methods that were used to solve the linear network could be extended to a loop network. Using the solution methods to solve a loop network could offer further insight into the hydraulic performance of the water distribution; and

iii. Assess the chlorine decay with respect to pipe material and its impact on hydraulic parameters in a non-intermittent/continuous flow water distribution system.

In addition to recommendations listed above, based on the results of this study and the current demands, Maralal water distribution system should be rehabilitated as follows:

i. PIPE0 (200mm diameter) with the updated roughness factor of 65.5 should be replaced immediately with a new pipe of the same diameter;

ii. PIPE2, PIPE3, PIPE4, PIPE5, PIPE6, PIPE7, PIPE8 and PIPE9 all with 100mm diameter to be upgraded to 150mm diameter pipes for an improved system performance; and

iii. A chlorine dosing point should be installed at NODE2 in order to ensure that residual chlorine doesn’t fall below the minimum levels(0.2mg/l) throughout the network.

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APPENDICES

APPENDIX A: TABLES

Table A1: County Zone Nodes Pressure(m) and Pipes Headloss(m)

DAY 1						**DAY 2**						**DAY 3**					
PRESSURE(m)		**HEADLOSS**	**PRESSURE(m)**		**HEADLOSS**	**PRESSURE(m)**		**HEADLOSS**	**PRESSURE(m)**		**HEADLOSS**	**PRESSURE(m)**		**HEADLOSS**	**PRESSURE(m)**		**HEADLOSS**
NODE 2	NODE 3	**PIPE 1(m)**	NODE 3	NODE 4	**PIPE 2(m)**	NODE 2	NODE 3	**PIPE 1(m)**	NODE 3	NODE 4	**PIPE 2(m)**	NODE 2	NODE 3	**PIPE 1(m)**	NODE 3	NODE 4	**PIPE 2(m)**
48.55	45.45	6.10	45.45	12.47	5.98	48.46	45.65	5.81	45.65	12.55	6.10	48.36	45.06	5.91	45.06	12.67	5.78
48.44	44.44	7.00	44.44	11.13	6.31	48.43	44.54	6.90	44.54	11.24	6.30	48.34	44.64	6.70	44.64	11.24	6.40
48.23	43.10	8.13	43.10	9.74	6.36	48.13	43.20	7.93	43.20	9.82	6.38	48.03	43.16	7.87	43.16	9.69	6.47
47.18	42.00	8.18	42.00	8.50	6.50	47.08	42.10	7.98	42.10	8.63	6.47	47.00	42.12	7.88	42.12	8.63	6.49
46.21	41.00	8.21	41.00	7.47	6.53	46.13	41.10	8.11	41.10	7.56	6.54	46.11	41.15	7.96	41.15	7.56	6.58
46.05	40.56	8.49	40.56	6.96	6.60	46.00	40.66	8.34	40.66	7.00	6.66	45.95	40.66	8.29	40.66	7.00	6.66
45.85	40.12	8.73	40.12	6.32	6.80	45.76	40.22	8.53	40.22	6.41	6.81	45.65	40.32	8.33	40.32	6.39	6.93
45.18	39.15	9.03	39.15	5.05	7.10	45.08	39.25	8.83	39.25	5.35	6.90	45.00	39.25	8.75	39.25	5.20	7.05
44.10	38.00	9.10	38.00	3.88	7.12	44.10	38.10	9.00	38.10	4.02	7.08	43.95	38.10	8.85	38.10	4.02	7.08
43.56	37.00	9.56	37.00	2.79	7.21	43.44	37.10	9.36	37.10	3.00	7.10	43.36	37.10	9.26	37.10	3.00	7.17
42.88	36.00	9.88	39.00	1.76	7.24	42.71	35.95	9.83	35.95	1.62	7.19	42.68	35.00	9.71	35.00	1.86	7.11
41.96	35.00	9.96	35.00	0.70	7.30	41.82	35.00	9.86	35.00	0.60	7.24	41.74	34.00	9.88	34.00	0.65	7.23
DAY 4						**DAY 5**						**DAY 6**					
48.86	45.54	6.32	45.54	12.50	6.04	49.16	45.56	6.60	45.56	12.48	6.08	48.96	45.50	6.46	45.50	12.35	6.15
48.74	44.55	7.19	44.55	11.21	6.34	48.75	44.54	7.21	44.54	11.25	6.29	48.65	44.55	7.10	44.55	11.20	6.35
48.13	43.21	7.92	43.21	9.81	6.40	48.23	43.20	8.03	43.20	9.83	6.37	48.33	43.24	8.09	43.24	9.85	6.39
47.10	42.12	7.98	42.12	8.64	6.48	47.18	42.11	8.07	42.11	8.65	6.46	47.28	42.12	8.16	42.12	8.55	6.57
46.23	41.12	8.11	41.12	7.60	6.52	46.31	41.10	8.21	41.10	7.58	6.52	46.38	41.09	8.29	41.09	7.18	6.91
46.10	40.56	8.54	40.56	7.00	6.56	46.11	40.66	8.45	40.66	7.02	6.64	46.18	40.68	8.50	40.68	6.72	6.96
45.77	40.32	8.45	40.32	6.41	6.91	45.85	40.22	8.63	40.22	6.43	6.79	45.85	40.20	8.65	40.20	6.18	7.02
45.18	39.24	8.94	39.24	5.35	6.89	45.18	39.25	8.93	39.25	5.37	7.88	45.28	39.30	8.98	39.30	5.22	7.08
44.12	38.15	8.97	38.15	4.22	8.93	44.10	38.10	9.00	38.10	4.05	7.05	44.18	38.12	9.06	38.12	4.00	7.12
43.47	37.10	9.37	37.10	3.02	7.08	43.56	37.02	9.54	37.02	2.95	7.07	43.60	37.01	9.50	37.01	2.87	7.23
42.79	35.96	9.83	35.96	1.85	7.11	42.88	35.96	9.92	35.96	1.85	7.11	42.90	35.96	9.94	35.96	1.60	7.36
41.93	35.08	9.85	35.08	0.94	7.14	41.96	34.98	9938	34.98	0.76	7.22	42.20	35.10	10.10	35.10	0.70	7.40

Table A2: PCEA Zone Nodes Pressure (m)

	NODE 2								NODE 6					NODE 7							NODE 8		
Day 1	**Day 2**	**Day 3**	**Day 4**	**Day 5**	**Day 6**	**Day 1**	**Day 2**	**Day 3**	**Day 4**	**Day 5**	**Day 6**	**Day 1**	**Day 2**	**Day 3**	**Day 4**	**Day 5**	**Day 6**	**Day 1**	**Day 2**	**Day 3**	**Day 4**	**Day 5**	**Day 6**
57.75	58.00	57.92	57.70	58.00	57.93	48.12	48.23	47.96	48.10	47.96	47.95	31.20	30.43	30.10	31.21	30.08	30.00	30.11	29.23	29.00	30.10	29.00	29.10
56.43	57.43	57.36	56.44	57.40	57.37	46.52	47.45	47.35	46.50	47.25	47.36	29.48	29.44	29.44	29.50	29.34	29.38	28.18	28.18	28.18	28.15	28.15	28.18
55.21	56.22	56.15	55.20	56.18	56.14	45.18	46.15	46.10	45.15	46.00	46.11	28.10	28.10	28.10	28.02	27.96	28.11	26.72	26.72	26.72	26.70	26.70	26.72
54.39	55.39	55.40	54.40	55.42	55.42	44.16	45.12	45.10	44.17	45.10	45.08	26.92	26.92	26.92	26.89	26.92	26.92	25.32	25.32	25.32	25.33	25.30	25.32
53.75	54.76	54.77	53.76	54.76	54.75	43.46	44.35	44.32	43.45	44.32	44.32	26.00	26.01	26.00	26.10	26.00	26.00	24.03	24.03	24.03	24.03	24.22	24.03
52.47	53.47	53.38	52.50	53.40	53.38	42.12	42.90	42.86	42.15	42.86	42.84	24.43	24.43	24.42	24.44	24.42	24.42	22.36	22.36	22.36	22.35	22.36	22.42
50.38	51.40	51.34	50.40	51.64	51.35	40.00	40.00	39.98	40.02	39.98	39.58	21.25	21.27	21.25	21.26	21.20	21.12	19.06	19.06	19.06	19.00	19.06	19.06
49.41	50.41	50.40	49.42	50.42	50.41	38.92	38.92	38.92	38.92	38.72	38.55	20.01	20.05	20.00	20.00	21.84	19.96	17.75	17.75	17.75	17.65	18.90	17.76
47.94	48.96	48.95	47.88	48.75	48.92	37.13	37.13	37.13	37.15	36.93	37.03	18.05	17.99	18.00	18.20	17.99	17.75	15.40	15.40	15.40	15.41	15.00	15.40
46.81	47.72	47.75	46.75	47.65	47.70	35.74	35.74	35.75	35.70	35.75	35.75	16.17	15.92	16.00	16.18	16.65	16.00	13.26	13.26	13.26	13.36	13.50	13.26
45.32	46.30	46.31	45.30	46.32	46.32	34.10	34.10	34.13	34.11	34.10	34.10	14.72	14.02	15.20	14.02	14.95	15.20	12.10	11.30	12.10	11.00	11.72	12.13
44.15	45.10	45.00	44.10	44.98	44.99	32.84	32.84	32.64	32.85	32.64	32.56	13.42	12.56	13.61	12.70	13.41	13.51	10.30	9.80	10.30	9.52	10.00	10.30

Table A3: PCEA Zone Pipes Headloss Measured (m)

PIPE 7						PIPE 8						PIPE 9					
Day 1	**Day 2**	**Day 3**	**Day 4**	**Day 5**	**Day 6**	**Day 1**	**Day 2**	**Day 3**	**Day 4**	**Day 5**	**Day 6**	**Day 1**	**Day 2**	**Day 3**	**Day 4**	**Day 5**	**Day 6**
14.63	14.77	14.96	14.60	15.04	14.98	20.92	21.80	21.86	20.89	21.88	21.95	3.09	3.2	3.1	3.11	3.08	2.9
14.91	14.98	15.01	14.94	15.15	15.01	21.04	22.01	21.91	21.00	21.91	21.98	3.30	3.26	3.26	3.35	3.19	3.2
15.03	15.07	15.05	15.05	15.18	15.03	21.08	22.05	22.00	21.13	22.04	22.00	3.38	3.38	3.38	3.32	3.26	3.39
15.23	15.27	15.30	15.23	15.32	15.34	21.24	22.20	22.18	21.28	22.18	22.16	3.60	3.6	3.6	3.56	3.62	3.6
15.29	15.41	15.45	15.31	15.44	15.43	21.46	22.34	22.32	21.35	22.32	22.32	3.97	3.98	3.97	4.07	3.78	3.97
15.35	15.57	15.52	15.35	15.54	15.54	21.69	22.47	22.44	21.71	22.44	22.42	4.07	4.07	4.06	4.09	4.06	4
15.38	16.40	16.36	15.38	16.66	16.77	22.75	22.73	22.73	22.76	22.78	22.46	4.19	4.21	4.19	4.26	4.14	4.06
15.49	16.49	16.48	15.50	16.70	16.86	22.91	22.87	22.92	22.92	20.88	22.59	4.26	4.3	4.25	4.35	4.94	4.2
15.81	16.83	16.82	15.73	16.82	16.89	23.08	23.14	23.13	22.95	22.94	23.28	4.65	4.59	4.6	4.79	4.99	4.35
16.07	16.98	17.00	16.05	16.90	16.95	23.57	23.82	23.75	23.52	23.10	23.75	4.91	4.66	4.74	4.82	5.15	4.74
16.22	17.20	17.18	16.19	17.22	17.22	23.38	24.08	22.93	24.09	23.15	22.90	4.62	4.72	5.1	5.02	5.23	5.07
16.31	17.26	17.36	16.25	17.34	17.43	23.42	24.28	23.03	24.15	23.23	23.05	5.12	4.76	5.31	5.18	5.41	5.21

Table A4: Town Zone 1 Nodes Pressure (m)

		NODE 1					NODE 2						NODE 3						NODE 4				
Day 1	**Day 2**	**Day 3**	**Day 4**	**Day 5**	**Day 6**	**Day 1**	**Day 2**	**Day 3**	**Day 4**	**Day 5**	**Day 6**	**Day 1**	**Day 2**	**Day 3**	**Day 4**	**Day 5**	**Day 6**	**Day 1**	**Day 2**	**Day 3**	**Day 4**	**Day 5**	**Day 6**
58.2	48.8	57.7	58.60	57.73	58.7	48.7	48.7	48.65	49.17	48.65	48.75	24.12	23.27	24.10	23.17	24.10	24.14	26.20	25.28	26.21	26.20	26.22	26.19
57.8	47.9	57.2	58.08	57.18	58.18	47.99	47.89	47.82	48.90	47.82	47.92	23.15	22.35	23.25	22.35	23.25	23.16	25.00	24.26	25.10	25.02	25.11	25.09
57.5	57.7	56.8	57.66	56.76	57.75	47.48	47.25	47.18	48.21	47.18	47.28	22.33	21.56	22.34	21.56	22.34	22.33	24.10	23.44	24.15	24.10	24.15	24.10
57.3	57.5	56.5	57.50	56.52	57.49	46.97	46.67	46.25	47.67	46.25	46.27	21.25	20.43	21.26	20.43	21.26	21.27	23.01	22.00	23.00	23.01	23.00	23.01
56.9	57	56	57.00	56.04	57.01	45.98	45.58	45.68	46.58	45.68	45.68	20.56	19.18	20.50	19.18	20.50	20.55	22.16	20.12	22.16	22.16	22.16	22.16
56.6	56.7	55.8	56.69	55.79	56.78	45.71	45.22	45.4	46.22	45.4	45.41	20.16	18.54	20.15	18.54	20.15	20.16	21.28	19.15	21.28	21.28	21.28	21.28
56.3	56.5	55.5	56.45	55.49	56.49	45.25	44.95	45.05	45.95	45.05	45	19.63	17.71	19.03	17.71	19.03	19.63	20.56	18.23	20.14	20.56	20.14	20.56
55.8	55.9	55	55.88	54.98	55.94	44.67	44.35	44.37	45.35	44.37	44.37	18.76	17.00	18.75	17.00	18.75	18.76	19.45	17.41	19.45	19.45	19.45	19.45
55.3	55.5	54.5	55.47	54.51	55.53	43.95	43.86	43.85	44.86	43.85	43.85	18.05	16.32	18.04	16.32	18.04	18.05	18.42	16.52	18.42	18.42	18.42	18.42
55	55.2	54.2	55.20	54.2	55.21	43.62	43.42	43.42	44.42	43.42	43.42	17.44	15.75	17.37	15.75	17.37	17.44	17.56	15.48	17.56	17.56	17.56	17.56
54.8	55	53.9	55.95	53.92	54.93	43.41	43.1	43.1	44.11	43.1	43.11	17.00	15.10	16.98	15.10	16.98	17.00	17.00	14.79	17.12	17.00	17.12	17.02
54.4	54.5	53.6	55.46	53.55	54.57	42.91	42.5	42.45	43.45	42.45	42.51	16.08	14.24	16.18	14.22	16.18	16.10	16.00	13.86	16.02	16.00	16.02	16.01

Table A5: Town Zone 1 Pipes Headloss (m)

PIPE 1							PIPE 3							PIPE 4			
Day 1	**Day 2**	**Day 3**	**Day 4**	**Day 5**	**Day 6**	**Day 1**	**Day 2**	**Day 3**	**Day 4**	**Day 5**	**Day 6**	**Day 1**	**Day 2**	**Day 3**	**Day 4**	**Day 5**	**Day 6**
12.53	12.90	12.08	12.43	12.08	12.95	22.58	23.4	22.55	24	22.55	22.61	7.92	7.99	7.89	6.97	7.88	7.95
12.79	13.19	12.36	12.18	12.36	13.26	22.84	23.5	22.57	24.55	22.57	22.76	8.15	8.09	8.15	7.33	8.14	8.07
12.98	13.41	12.58	12.45	12.58	13.47	23.15	23.7	22.84	24.65	22.84	22.95	8.23	8.12	8.19	7.46	8.19	8.23
13.35	13.83	13.27	12.83	13.27	14.22	23.72	24.2	22.99	25.24	22.99	23.00	8.24	8.43	8.26	7.42	8.26	8.26
13.87	14.42	13.36	13.42	13.36	14.33	23.42	24.4	23.18	25.4	23.18	23.13	8.4	9.06	8.34	7.02	8.34	8.39
13.89	14.47	13.39	13.47	13.39	14.37	23.55	24.7	23.25	25.68	23.25	23.25	8.88	9.39	8.87	7.26	8.87	8.88
14.03	14.50	13.44	13.50	13.44	14.49	23.62	25.2	24.02	26.24	24.02	23.37	9.07	9.48	8.89	7.15	8.89	9.07
14.11	14.53	13.61	13.53	13.61	14.57	23.91	25.4	23.62	26.35	23.62	23.61	9.31	9.59	9.3	7.55	9.3	9.31
14.37	14.61	13.66	13.61	13.66	14.68	23.90	25.5	23.81	26.54	23.81	23.80	9.63	9.8	9.62	7.9	9.62	9.63
14.38	14.78	13.78	13.78	13.78	14.79	24.18	25.7	24.05	26.67	24.05	23.98	9.88	10.27	9.81	8.19	9.81	9.88
14.42	14.85	13.82	14.84	13.82	14.82	24.41	26	24.12	27.01	24.12	24.11	10	10.31	9.86	8.1	9.86	9.98
14.44	14.96	14.10	15.01	14.10	15.06	24.83	26.3	24.27	27.23	24.27	24.41	10.1	10.38	10.2	8.22	10.16	10.09

Table A6: Town Zone 2 Nodes Pressure (m)

NODE 2						NODE 3						NODE 7						NODE 9					
Day 1	**Day 2**	**Day 3**	**Day 4**	**Day 5**	**Day 6**	**Day 1**	**Day 2**	**Day 3**	**Day 4**	**Day 5**	**Day 6**	**Day 1**	**Day 2**	**Day 3**	**Day 4**	**Day 5**	**Day 6**	**Day 1**	**Day 2**	**Day 3**	**Day 4**	**Day 5**	**Day 6**
59.74	59.65	58.65	59.55	60	59.67	48.06	48.06	47.06	48.01	48.15	47.96	17.84	18.00	17.00	17.85	17.84	18.10	18.00	18.11	17.20	17.91	18.10	17.98
58.96	58.86	57.86	58.72	59.86	58.85	47.07	47.07	46.07	47.11	47.63	47.1	16.67	16.97	16.87	16.87	16.90	16.92	16.71	17.03	17.03	16.93	17.00	16.70
58.06	58	57	58.12	59.12	58.01	46.02	46.02	45.02	46.12	46.14	46	15.50	15.45	15.35	15.38	15.37	15.36	15.50	14.80	14.80	15.31	14.85	14.81
56.93	56.88	55.88	56.80	57.88	56.86	44.82	44.82	43.82	44.83	44.77	44.83	14.15	14.00	14.00	14.01	13.92	14.10	13.50	13.26	13.26	13.85	13.36	13.20
55.94	55.74	54.74	55.75	56.74	55.75	43.56	43.56	42.56	43.60	43.56	43.58	12.86	12.56	12.48	12.56	12.58	12.57	12.12	11.45	11.45	12.26	11.50	11.40
54.1	54.9	53.01	54.89	55.9	54.91	41.55	42.55	40.55	42.65	42.55	42.65	10.81	11.42	10.45	11.54	11.48	11.46	10.65	10.50	10.50	11.16	10.21	10.22
53.91	53.85	52.85	53.80	54.85	53.8	41.32	41.32	40.32	41.35	41.34	41.33	10.05	10.15	10.16	10.17	10.05	10.09	9.00	8.96	8.96	9.75	8.74	8.71
52.67	52.68	51.68	52.70	53.69	52.78	40	40	39	40.11	40.12	40.1	8.50	8.55	8.78	8.85	8.56	8.45	7.05	7.21	7.21	8.25	7.17	7.00
51.78	51.69	50.69	51.70	52.7	51.7	39	39	38	39.00	38.95	38.96	7.35	7.46	7.66	7.48	7.35	7.25	5.72	6.00	6.00	6.35	5.82	5.72
50.98	50.68	49.68	50.63	51.68	50.78	38.02	37.9	36.9	37.88	37.84	37.89	6.30	6.23	6.21	6.25	6.10	6.03	4.22	4.62	4.52	4.85	4.32	4.32
50.26	50.06	49.06	50.05	51.13	50.16	37.15	37.15	36.15	37.22	37.21	37	5.32	5.28	5.40	5.30	5.22	5.10	3.04	3.52	3.52	3.52	3.36	3.31
49.11	49	48	49.12	50.2	49.32	35.85	35.85	34.85	35.85	35.77	35.77	4.00	3.95	4.00	3.85	3.41	3.80	1.60	2.00	2.00	1.91	1.40	1.69

Table A7: Town Zone 2 Pipes Headloss (m)

PIPE 1						**PIPE 6**							**PIPE 5**				
Day 1	**Day 2**	**Day 3**	**Day 4**	**Day 5**	**Day 6**	**Day 1**	Day 2	Day 3	Day 4	Day 5	Day 6	Day 1	Day 2	Day 3	Day 4	Day 5	Day 6
14.68	14.59	14.59	14.54	14.85	14.71	36.22	36.06	36.06	36.16	36.31	35.86	1.84	1.89	1.8	1.94	1.74	2.12
14.89	14.79	14.79	14.61	15.23	14.75	36.4	36.10	35.2	36.24	36.73	36.18	1.96	1.94	1.84	1.94	1.9	2.22
15.04	14.98	14.98	15.00	15.98	15.01	36.52	36.57	35.67	36.74	36.77	36.64	2	2.65	2.55	2.07	2.52	2.55
15.11	15.06	15.06	14.97	16.11	15.03	36.67	36.82	35.82	36.82	36.85	36.73	2.65	2.74	2.74	2.16	2.56	2.9
15.38	15.18	15.18	15.15	16.18	15.17	36.7	37.00	36.08	37.04	36.98	37.01	2.74	3.11	3.03	2.3	3.08	3.17
15.55	15.35	15.46	15.24	16.35	15.26	36.74	37.13	36.1	37.11	37.07	37.19	2.16	2.92	1.95	2.38	3.27	3.24
15.59	15.53	15.53	15.45	16.51	15.47	37.27	37.17	36.16	37.18	37.29	37.24	3.05	3.19	3.2	2.42	3.31	3.38
15.67	15.68	15.68	15.59	16.57	15.68	37.5	37.45	36.22	37.26	37.56	37.65	3.45	3.34	3.57	2.6	3.39	3.45
15.78	15.69	15.69	15.70	16.75	15.74	37.65	37.54	36.34	37.52	37.6	37.71	3.63	3.46	3.66	3.13	3.53	3.53
15.96	15.78	15.78	15.75	16.84	15.89	37.72	37.67	36.69	37.63	37.74	37.86	4.08	3.61	3.69	3.4	3.78	3.71
16.11	15.91	15.91	15.83	16.92	16.16	37.83	37.87	36.75	37.92	37.99	37.90	4.28	3.76	3.88	3.78	3.86	3.79
16.26	16.15	16.15	16.27	17.43	16.55	37.85	37.9	36.85	38	38.36	37.97	4.4	3.95	4	3.94	4.01	4.11

Table A8: County Zone Pipes Flow (m^3/hr)

PIPE 1						PIPE 2					
DAY 1	**DAY 2**	**DAY 3**	**DAY 4**	**DAY 5**	**DAY 6**	**DAY 1**	**DAY 2**	**DAY 3**	**DAY 4**	**DAY 5**	**DAY 6**
55.85	54.40	54.91	56.93	58.28	57.61	29.92	30.24	29.38	30.08	30.19	30.38
60.16	59.70	58.75	61.04	61.13	60.62	30.80	30.78	31.04	30.88	30.75	30.91
65.22	64.35	64.09	64.31	64.79	65.05	30.93	30.99	31.22	31.04	30.96	31.01
65.44	64.57	64.13	64.57	64.96	65.35	31.30	31.22	31.27	31.25	31.19	31.48
65.57	65.14	64.48	65.14	65.57	65.91	31.38	31.40	31.53	31.35	31.35	32.35
66.77	66.13	65.91	66.98	66.60	66.81	31.56	31.71	31.71	31.45	31.66	32.48
67.78	66.94	66.09	66.60	67.36	67.44	32.07	32.10	32.40	32.35	32.05	32.63
69.03	68.20	67.86	68.66	68.61	68.82	32.83	32.32	32.70	32.30	32.27	32.78
69.32	68.90	68.28	68.78	68.90	69.15	32.88	32.78	32.78	32.40	32.70	32.88
71.19	70.38	69.97	70.42	71.11	70.95	33.10	32.83	32.83	32.78	32.75	33.15
72.46	72.27	71.79	72.27	72.62	72.70	33.18	33.05	32.85	32.85	32.85	33.47
72.78	72.38	72.46	72.35	72.86	73.33	33.32	33.18	33.15	32.93	33.13	33.57

Table A9: PCEA Zone Pipes Flow (m^3/hr)

PIPE 7						PIPE 8						PIPE 9					
DAY 1	**DAY 2**	**DAY 3**	**DAY 4**	**DAY 5**	**DAY 6**	**DAY 1**	**DAY 2**	**DAY 3**	**DAY 4**	**DAY 5**	**DAY 6**	**DAY 1**	**DAY 2**	**DAY 3**	**DAY 4**	**DAY 5**	**DAY 6**
51.37	51.64	52.00	51.32	52.15	52.03	51.32	52.48	52.56	51.28	52.58	52.67	14.80	15.08	14.82	14.85	14.77	14.30
51.90	52.03	52.09	51.96	52.35	52.09	51.48	52.75	52.62	51.43	52.62	52.71	15.33	15.23	15.23	15.46	15.05	15.08
52.13	52.20	52.17	52.17	52.41	52.13	51.53	52.80	52.74	51.60	52.79	52.74	15.53	15.53	15.53	15.38	15.23	15.55
52.50	52.58	52.63	52.50	52.67	52.71	51.75	53.00	52.97	51.80	52.97	52.94	16.07	16.07	16.07	15.97	16.12	16.07
52.61	52.84	52.91	52.65	52.89	52.87	52.03	53.18	53.15	51.89	53.15	53.15	16.94	16.96	16.94	17.17	16.50	16.94
52.72	53.13	53.04	52.72	53.08	53.08	52.33	53.34	53.30	52.36	53.30	53.28	17.17	17.17	17.15	17.21	17.15	17.01
52.78	54.64	54.57	52.78	55.11	55.30	53.70	53.67	53.67	53.71	53.74	53.33	17.44	17.48	17.44	17.60	17.33	17.15
52.98	54.80	54.79	53.00	55.18	55.46	53.90	53.85	53.92	53.92	51.27	53.50	17.60	17.69	17.57	17.80	19.06	17.46
53.57	55.41	55.39	53.42	55.39	55.52	54.12	54.20	54.18	53.95	53.94	54.37	18.45	18.32	18.34	18.75	19.17	17.80
54.05	55.68	55.71	54.01	55.53	55.62	54.74	55.05	54.96	54.67	54.14	54.96	19.00	18.47	18.64	18.81	19.49	18.64
54.32	56.07	56.03	54.26	56.10	56.10	54.50	55.37	53.93	55.39	54.21	53.89	18.38	18.60	19.39	19.23	19.66	19.33
54.48	56.17	56.35	54.37	56.31	56.47	54.55	55.62	54.06	55.46	54.31	54.08	19.43	18.68	19.82	19.56	20.02	19.62

Table A10: Town Zone 1 Pipe Flow (m^3/hr)

PIPE 1						PIPE 3						PIPE 4					
DAY 1	DAY 2	DAY 3	DAY 4	DAY 5	DAY 6	DAY 1	DAY 2	DAY 3	DAY 4	DAY 5	DAY 6	DAY 1	DAY 2	DAY 3	DAY 4	DAY 5	DAY 6
82.38	83.69	80.77	82.03	80.77	83.86	63.95	65.24	63.91	66.10	63.91	64.00	24.58	24.69	24.53	22.94	24.51	24.63
83.30	84.70	81.78	81.13	81.78	84.94	64.35	65.41	63.94	66.91	63.94	64.23	24.96	24.86	24.96	23.57	24.94	24.83
83.97	85.46	82.56	82.10	82.56	85.67	64.82	65.63	64.35	67.06	64.35	64.52	25.09	24.91	25.03	23.80	25.03	25.09
85.25	86.89	84.98	83.44	84.98	88.21	65.68	66.45	64.58	67.92	64.58	64.59	25.11	25.42	25.14	23.73	25.14	25.14
87.03	88.88	85.29	85.49	85.29	88.58	65.23	66.69	64.87	68.15	64.87	64.79	25.37	26.43	25.27	23.03	25.27	25.35
87.10	89.04	85.39	85.67	85.39	88.71	65.42	67.10	64.97	68.56	64.97	64.97	26.14	26.94	26.13	23.45	26.13	26.14
87.57	89.14	85.56	85.77	85.56	89.11	65.53	67.92	66.13	69.36	66.13	65.15	26.44	27.08	26.16	23.26	26.16	26.44
87.84	89.24	86.15	85.87	86.15	89.37	65.96	68.08	65.53	69.52	65.53	65.51	26.82	27.25	26.80	23.95	26.80	26.82
88.71	89.51	86.32	86.15	86.32	89.74	65.95	68.35	65.81	69.79	65.81	65.80	27.31	27.57	27.30	24.54	27.30	27.31
88.74	90.07	86.72	86.72	86.72	90.10	66.36	68.54	66.17	69.97	66.17	66.07	27.69	28.28	27.59	25.03	27.59	27.69
88.88	90.30	86.86	90.27	86.86	90.20	66.70	69.02	66.27	70.45	66.27	66.26	27.88	28.34	27.66	24.88	27.66	27.85
88.94	90.66	87.81	90.82	87.81	90.99	67.32	69.39	66.50	70.76	66.50	66.70	28.00	28.44	28.12	25.08	28.12	28.01

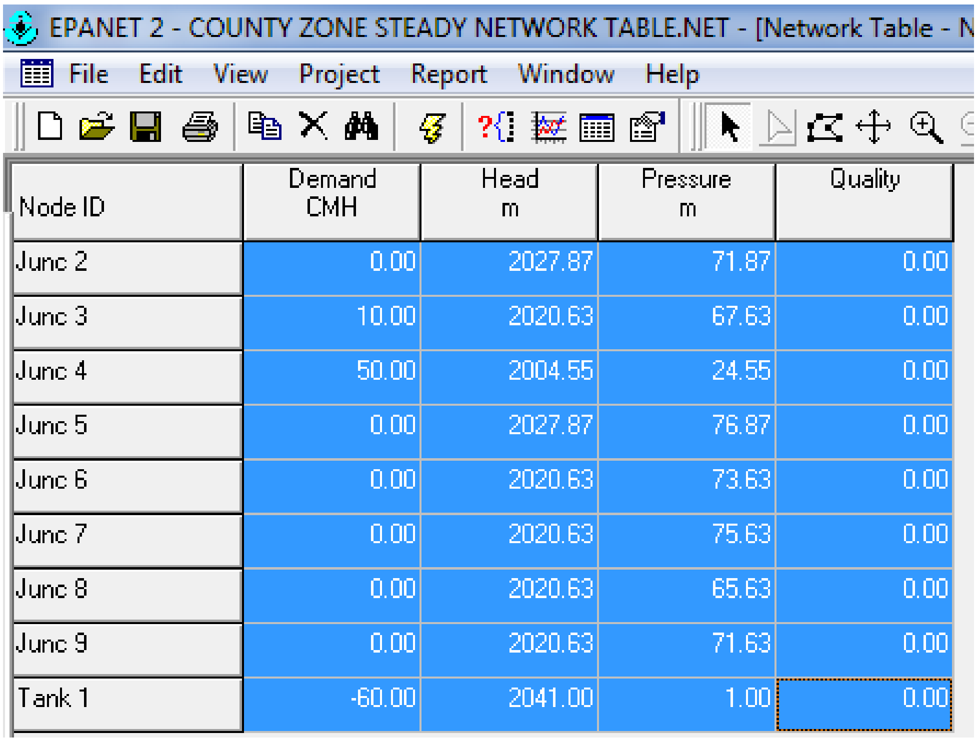
Table A11: Town Zone 2 Pipes Flow (m^3/hr)

PIPE 1						PIPE 5						PIPE 6					
DAY 1	DAY 2	DAY 3	DAY 4	DAY 5	DAY 6	DAY 1	DAY 2	DAY 3	DAY 4	DAY 5	DAY 6	DAY 1	DAY 2	DAY 3	DAY 4	DAY 5	DAY 6
89.74	89.44	89.44	89.28	90.30	89.84	21.57	21.88	21.31	22.19	20.93	23.28	58.30	58.16	58.16	58.25	58.38	57.98
90.43	90.10	90.10	89.51	91.54	89.97	22.32	22.19	21.57	22.19	21.94	23.87	58.45	58.19	57.41	58.31	58.74	58.26
90.92	90.72	90.72	90.79	93.95	90.82	22.56	26.26	25.72	22.98	25.56	25.72	58.56	58.60	57.82	58.75	58.77	58.66
91.15	90.99	90.99	90.69	94.36	90.89	26.26	26.74	26.74	23.52	25.78	27.57	58.69	58.82	57.95	58.82	58.84	58.74
92.02	91.38	91.38	91.28	94.58	91.34	26.74	28.63	28.23	24.33	28.48	28.93	58.71	58.97	58.18	59.01	58.95	58.98
92.57	91.93	92.28	91.57	95.11	91.64	23.52	27.67	22.25	24.78	29.42	29.27	58.75	59.08	58.19	59.07	59.03	59.14
92.70	92.51	92.51	92.25	95.62	92.31	28.33	29.03	29.08	25.01	29.61	29.95	59.20	59.12	58.25	59.13	59.22	59.18
92.96	92.99	92.99	92.70	95.80	92.99	30.28	29.76	30.85	25.99	30.00	30.28	59.40	59.36	58.30	59.20	59.45	59.53
93.31	93.02	93.02	93.05	96.36	93.18	31.13	30.33	31.26	28.73	30.66	30.66	59.53	59.44	58.40	59.42	59.49	59.58
93.88	93.31	93.31	93.21	96.64	93.66	33.15	31.03	31.40	30.04	31.81	31.49	59.59	59.55	58.70	59.51	59.61	59.71
94.36	93.72	93.72	93.47	96.89	94.52	34.02	31.72	32.27	31.81	32.18	31.86	59.68	59.72	58.76	59.76	59.82	59.74
94.83	94.48	94.48	94.86	98.46	95.74	34.53	32.58	32.80	32.53	32.84	33.28	59.70	59.74	58.84	59.83	60.13	59.80

Table A25: Simulation of the Dynamic System Performance Data

PIPE ID	LENGTH(m)	DIAMETER(mm)	ROUGHNESS	BULK COEFF	WALL COEFF	INITIAL CHLORINE(mg/l)	STATUS
PIPE0	2200	200	65.5	-1	-1.55	1	Open
PIPE1	874	150	117	-1	-1.55	1	Open
PIPE2	396	100	120	-1	-1.55	1	Open
PIPE3	355	100	116	-1	-1.55	1	Open
PIPE4	755	100	117	-1	-1.55	1	Open
PIPE5	220	100	120	-1	-1.55	1	Open
PIPE6	676	100	118	-1	-1.55	1	Open
PIPE7	356	100	119	-1	-1.55	1	Open
PIPE8	510	100	120	-1	-1.55	1	Open
PIPE9	754	100	119	-1	-1.55	1	Open

APPENDIX B: FIGURES



Node ID	Demand CMH	Head m	Pressure m	Quality
Junc 2	0.00	2027.87	71.87	0.00
Junc 3	10.00	2020.63	67.63	0.00
Junc 4	50.00	2004.55	24.55	0.00
Junc 5	0.00	2027.87	76.87	0.00
Junc 6	0.00	2020.63	73.63	0.00
Junc 7	0.00	2020.63	75.63	0.00
Junc 8	0.00	2020.63	65.63	0.00
Junc 9	0.00	2020.63	71.63	0.00
Tank 1	-60.00	2041.00	1.00	0.00

EPANET 2 - COUNTY ZONE STEADY NETWORK TABLE.NET - [Network Table - Links]

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Link ID	Flow CMH	Velocity m/s	Unit Headloss m/km	Friction Factor	Reaction Rate mg/L/d	Quality	Status
Pipe 2	60.00	0.94	8.28	0.027	0.00	0.00	Open
Pipe 3	50.00	1.77	40.61	0.025	0.00	0.00	Open
Pipe P	60.00	0.53	5.97	0.083	0.00	0.00	Open
Pipe 5	0.00	0.00	0.00	0.000	0.00	0.00	Closed
Pipe 6	0.00	0.00	0.00	0.000	0.00	0.00	Closed
Pipe 7	0.00	0.00	0.00	0.000	0.00	0.00	Open
Pipe 8	0.00	0.00	0.00	0.000	0.00	0.00	Closed
Pipe 9	0.00	0.00	0.00	0.000	0.00	0.00	Open
Pipe 10	0.00	0.00	0.00	0.000	0.00	0.00	Open
Pipe 1	0.00	0.00	0.00	0.000	0.00	0.00	Closed

Figure B1: County Zone Nodes and Links Results for steady state

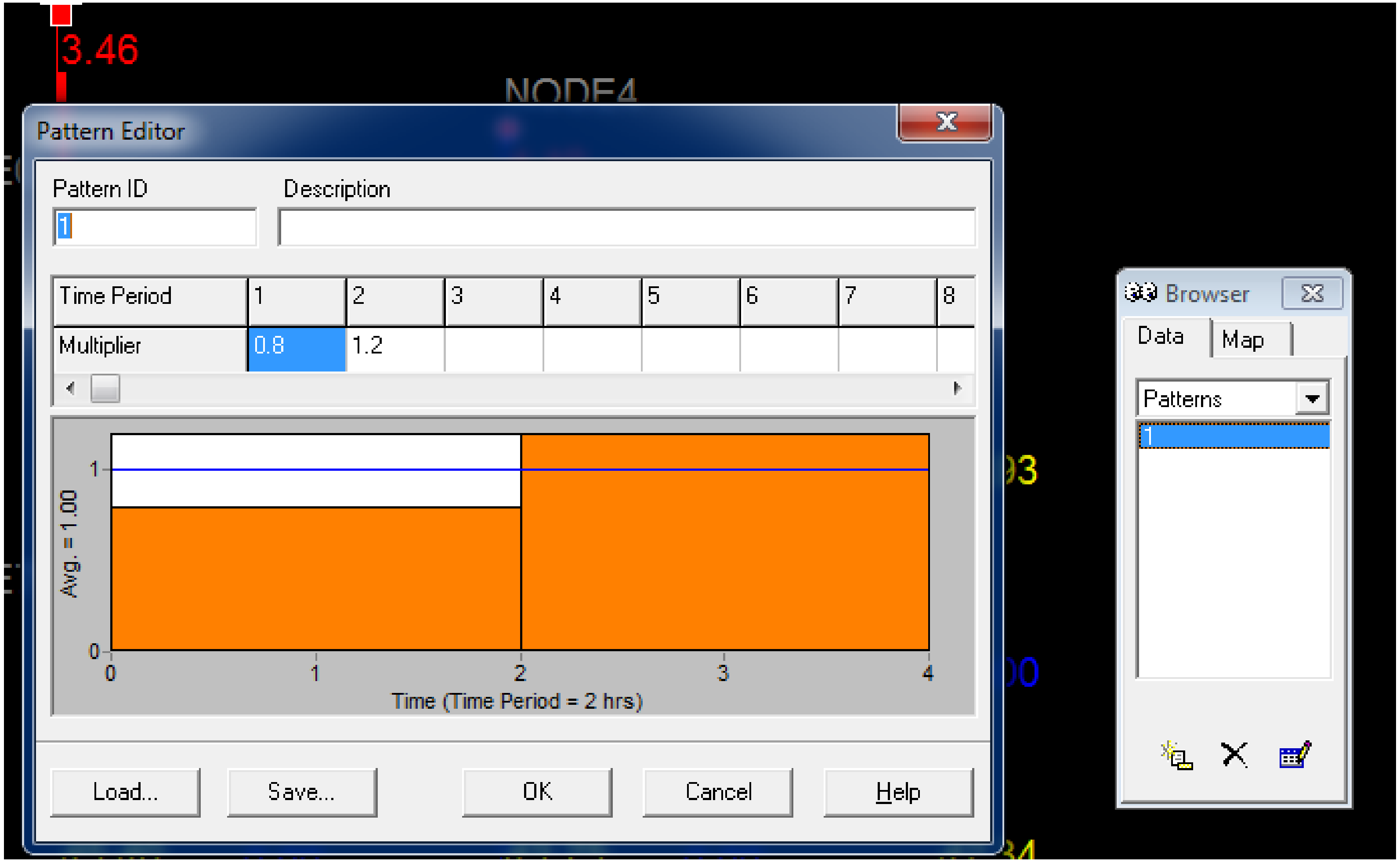

Figure B2: Time Pattern Multipliers

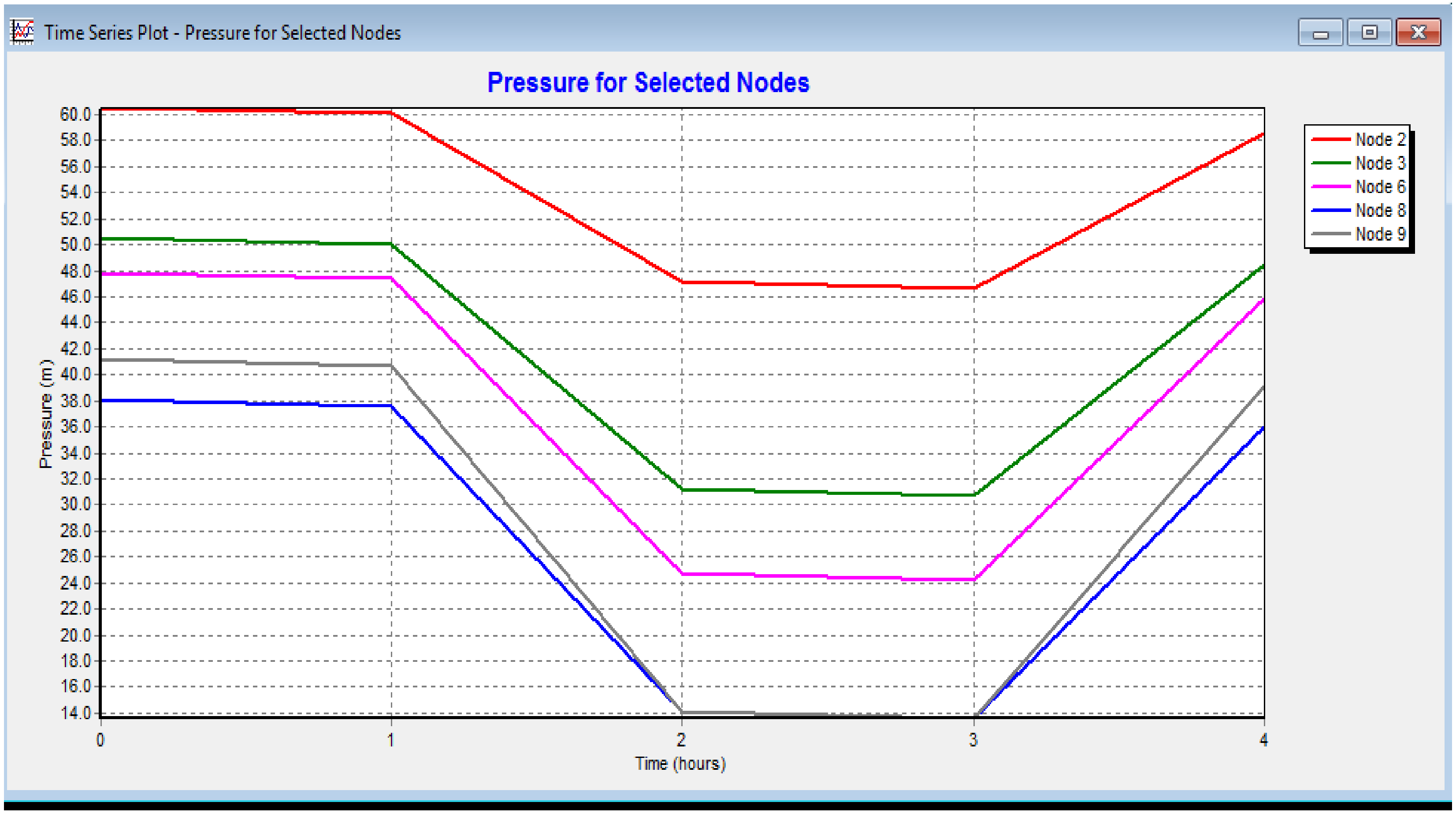

Figure B3: Model Output for Pressure at Nodes Over Time

APPENDIX C: PUBLICATION

OPEN ACCESS

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Simulation of Maralal Water Flow Distribution Network using EPANET Model in Samburu County, Kenya

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Abstract: ***Majority of people in developing countries do not have access to clean and potable water due to inadequate supply and distribution system challenges. While the rationale of water distribution systems is to deliver to each consumer safe drinking water that is adequate in quality and quantity at an acceptable delivery pressure, this has been a major drawback for many distribution networks. In addition, the design spans of many urban and peri-urban water distribution networks managed by the Water Service Providers (WSPs) are being exceeded without augmentation. Maralal water distribution network is one of such distribution systems with poor system performance that has been the main contributor of high Non-Revenue Water (NRW). This coupled with significant mismatch between water supply and water demand makes Maralal Water and Sanitation Company to resort to hedging/intermittent flow leading to water rationing. One of the ways of predicting the flow dynamics within the distribution system is the use of hydraulic simulation models. This study therefore applied the Environmental Protection Agency Network (EPANET) simulation model to predict the dynamic state of the hydraulics and water quality behaviour for Maralal water distribution system operating over an extended period of time. The general objective was to simulate water flow for Maralal water distribution system using the EPANET model for efficient planning, operation and maintenance protocol for the system. The study focused on the steady state (static), extended period (dynamic), and water quality analyses. The model calibration results from four statistical criteria; Nash-Sutcliffe model efficiency coefficient (E), Sum of Squares Error (SSE), Percentage Bias (PB) and Root Mean Square Error (RMSS) of 0.99, 0.01, 0.05 and 0.03 respectively show that the model performed within acceptable range of the selected statistical criteria. The findings of this study were: The roughness coefficients for a water distribution network that contribute to erratic pressure-dependent flows can be determined at any time using the regression analysis of the measured head loss and flow rate, EPANET model can predict the steady and dynamic hydraulic parameters for the current and future water distribution systems and Chlorine decay with respect to pipe diameter impacts on hydraulic performance and quality of water in a distribution network. The results from this study would assist water service providers and managers to make informed decisions in relation to water distribution system planning, operation and maintenance to achieve the desired current and future water demands.***

Keywords: EPANET, Roughness factor, Simulation, Water distribution network

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I. INTRODUCTION

The purpose of any water distribution system is to deliver to each consumer safe drinking water that is adequate in quantity, delivery pressure and acceptable in terms of taste, odour and appearance. However, continued population growth has placed increasing demand upon existing water distribution systems. This growth has necessitated the need to analyze existing and design new water distribution systems. In addition, recent concern and awareness about the safety of drinking water has raised other concerns on the quality of water delivered in the existing and proposed municipal or city water distribution systems.

Water distribution networks present complex systems that include different types of pipes and sizes, diverse types of valves, Tanks and pumps. These networks require significant investments for construction, operation and maintenance. In this regard, the awareness of all hydraulic parameters in a water distribution system is an absolute prerequisite for rational planning of new networks and upgrading of existing system elements. If the system elements, their functions and hydraulic parameters are not known, numerous problems are likely to occur at some point in time during the operation of the system. For water distribution networks, the challenges are mainly due to low or high pressures in certain parts of the system, occurrence of system defects, such as leakage and increased energy consumption [20].

Water utility enterprises in developed countries have already started researching on strategic solutions for water distribution systems rationalization, and water consumption optimization by use of simulation models [9]. In order to meet regulatory requirements and customer expectations, Water Service Providers are globally faced with the challenge of understanding their water distribution systems. One of the ways to understand the water distribution systems could be achieved through the analysis of the flow dynamics in the distribution systems. Simulation of the flow can offer alternative options in addressing the water distribution systems' challenges. Models can be used to predict the dynamic state hydraulics and water quality behaviour for a drinking water distribution system operating over an extended period of time. The models can also be used as tools to assist in the planning, operation and maintenance decision making.