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

Paul Lolmingani, Benedict M. Mutua, David N. Kamau

***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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* Correspondence Author

Paul Lolmingani*, Managing Director, Maralal Water and Sanitation Co. Ltd., Maralal, Kenya.

Prof. Dr.-Ing. Benedict M. Mutua, Deputy Vice-Chancellor (Planning, Partnerships, Research and Innovation), Kibabii University, Kenya.

Dr. Eng. David N. Kamau, Senior Lecturer, Department of Civil & Environmental Engineering, Egerton University, Njoro, Kenya.

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.

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.

Over the years, the actual pipe roughness factor within water distribution networks has rarely been periodically determined as a prerequisite to establish and predict the hydraulic performance of the system. This has led to the design span of many urban and peri-urban water utilities in Kenya being exceeded without augmentation. Roughness within aged water distribution networks causes friction which result 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. This reduces the system capacity by adding more head loss into the system network. 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.

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. Results of this study are useful to the water resources managers in enhancing water service delivery. The main objective of this research 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.

1.1 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 longitude 36041' and latitude 01005' and at an elevation of 1940m as shown in Figure 1.1.

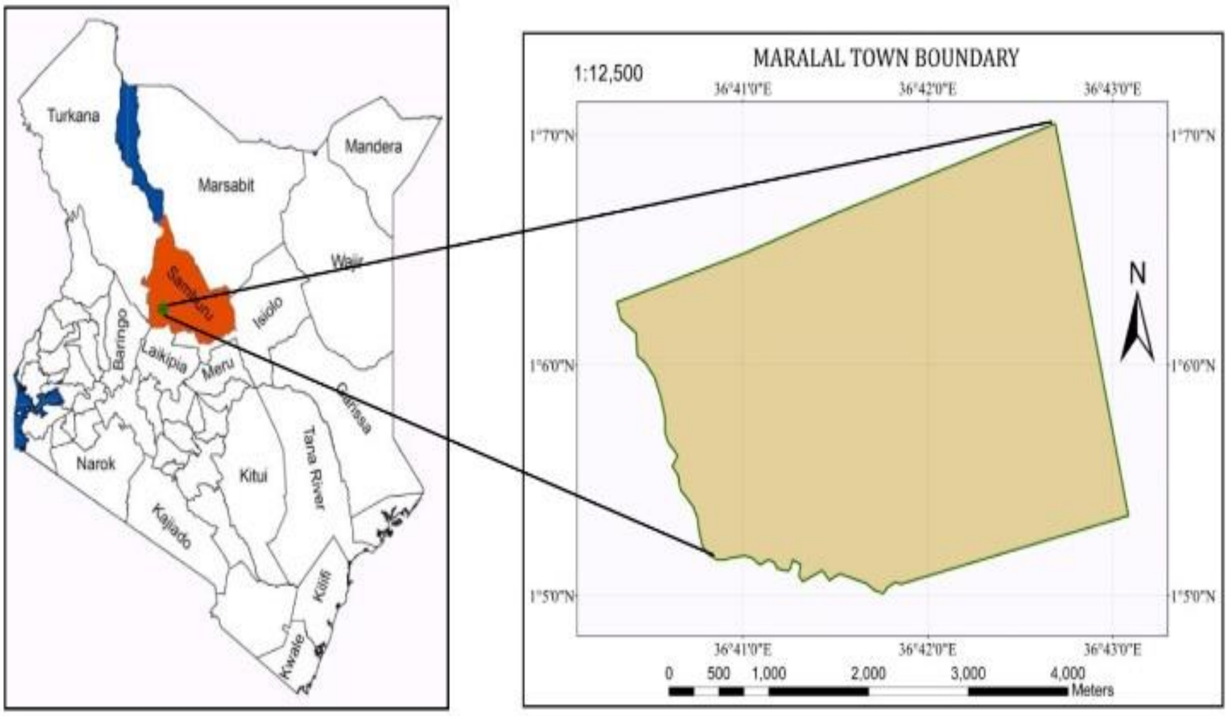

Fig. 1.1: Map of the Study Area (Source: Geocurrents)

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 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 Fig. 1.2.

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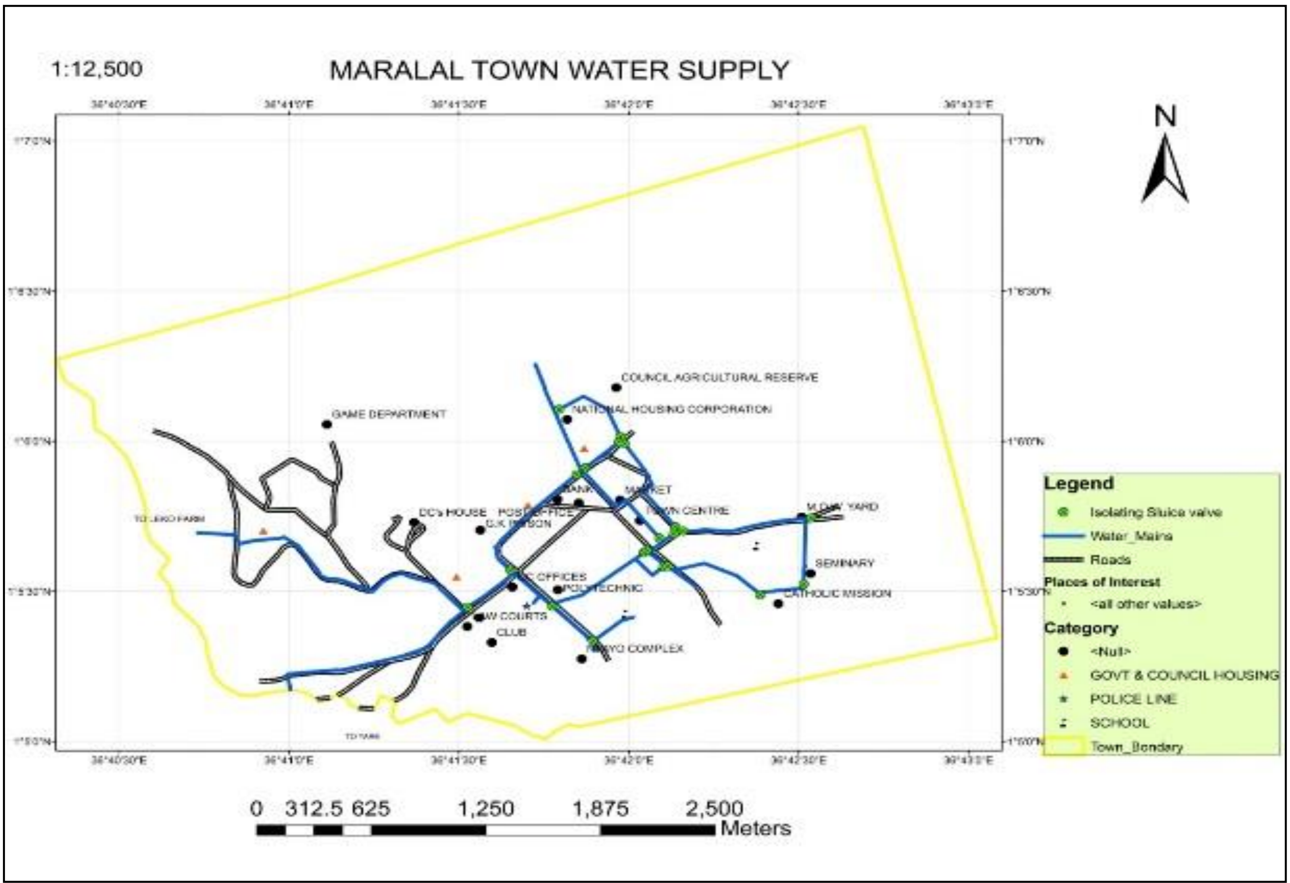

Fig. 1.2: Maralal Water Distribution Network

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.

II. LITERATURE REVIEW

Water is undoubtedly one of the essential commodities, with no other alternative, every living creature requires for survival [7]. However, 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 [5]. 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 [15]. For instance, [26] suggested the design and simulation of the water distribution network system by modeling, analyzing and performance evaluation through scenario investigation of the physical and hydraulic parameters. To many hydraulic engineers, it is common knowledge that the roughness factor in pipes change over time. Reference [11] found that change in roughness factor causes the head loss due to friction to increase by nearly 45%.

2.1 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 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 [28] for the computation of head loss is given as

$$h_{L}= \frac{C_f L}{C^{1.852} D^{4.87}} Q^{1.852} \qquad (2.1)$$

Where:

C = Hazen-Williams C-factor.

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

$$h_{L}=Aq^{B} \qquad (2.2)$$

Where:

h_L = head loss (m),

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

A = resistance coefficient,

B = flow exponent.

The effect of roughness factor on the hydraulics and the quality of water flowing through a pipe network is not well known in many water distribution systems. In some cases, the estimated roughness of old pipes, using optimization procedures based on measurements in operational water distribution system, can be quite demanding [14]. However, according to [21], hydraulic network analysis provides an option to understanding the malfunctioning of a water distribution system.

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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, firefighting flows [8]. Network analysis design applications include sizing of various types of facilities. Pipelines, pressure-regulating valves and ground Tanks can be sized using pressure and flow calculations resulting from hydraulic simulation. 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.

III. METHODOLGY

3.1 Materials

Materials used during the study are shown in Table 3.1 below.

Table 3.1: 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 Methods

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. Pressure was measured at eight (8) nodes within the Maralal water distribution network at an interval of 20minutes 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+).

3.2.1 Regression Analysis Of Calculated Headloss And Measured Flow Rate

Parameters A and B were determined by regression analysis of the measured head loss and flow rate using (2.2). 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.

3.2.2 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. After model calibration and validation, the model was run for different scenarios and the output for each scenario was obtained. 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 Fig. 3.1 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.

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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. Fig. 3.1 shows the Maralal virtual water distribution network in EPANET model environment.

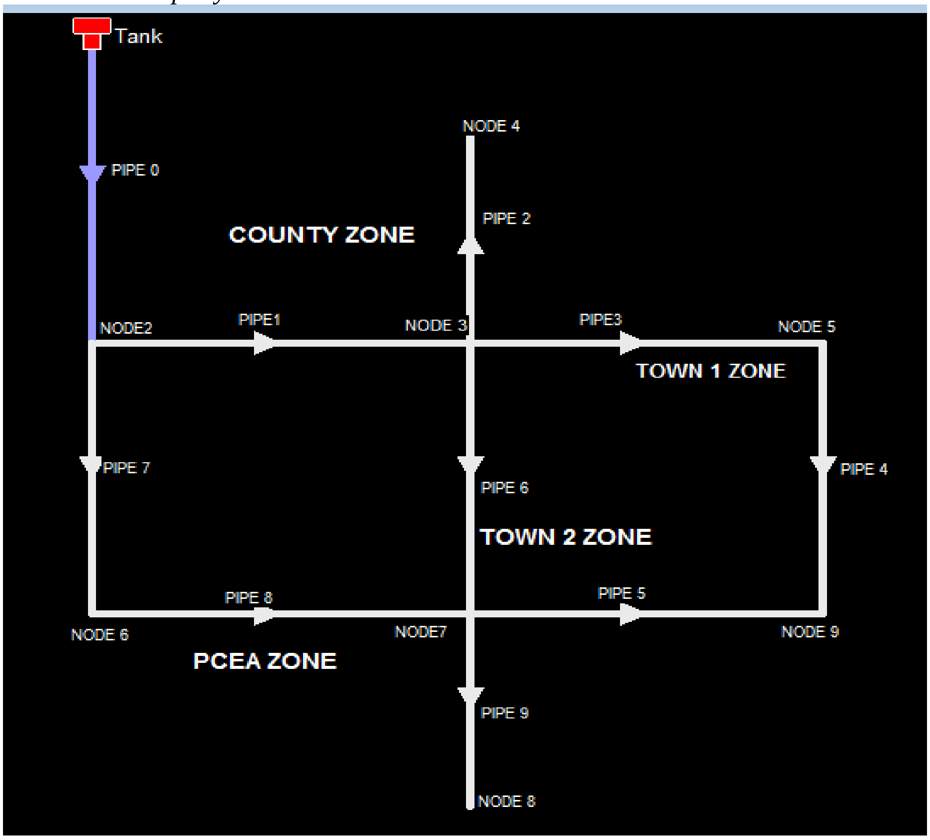

Fig. 3.1: Components of Maralal Water Distribution System using the EPANET Model

Figure 3.1 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.

3.2.3model Calibration

Calibration tests were performed with the purpose of testing the simulated data from the model against 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. The Nash-Sutcliffe model efficiency coefficient (E) was used to assess the predictive ability of the model. The efficiency coefficient equation is given as:

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

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.

3.2.4 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.

3.2.5 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.

3.3 impact of chlorine decay on system hydraulic performance.

Transport and decay of chlorine through the network were simulated using EPANET's water quality simulator.

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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 points in time. 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.

IV. 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. Headloss was calculated from measured pressure.

Table 4.1: 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 4.1, 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%.

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

4.2.1 Steady State 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 table as presented in Fig. 4.1. 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. Fig. 4.1 presents a summary of PCEA zone steady state pipe network results.

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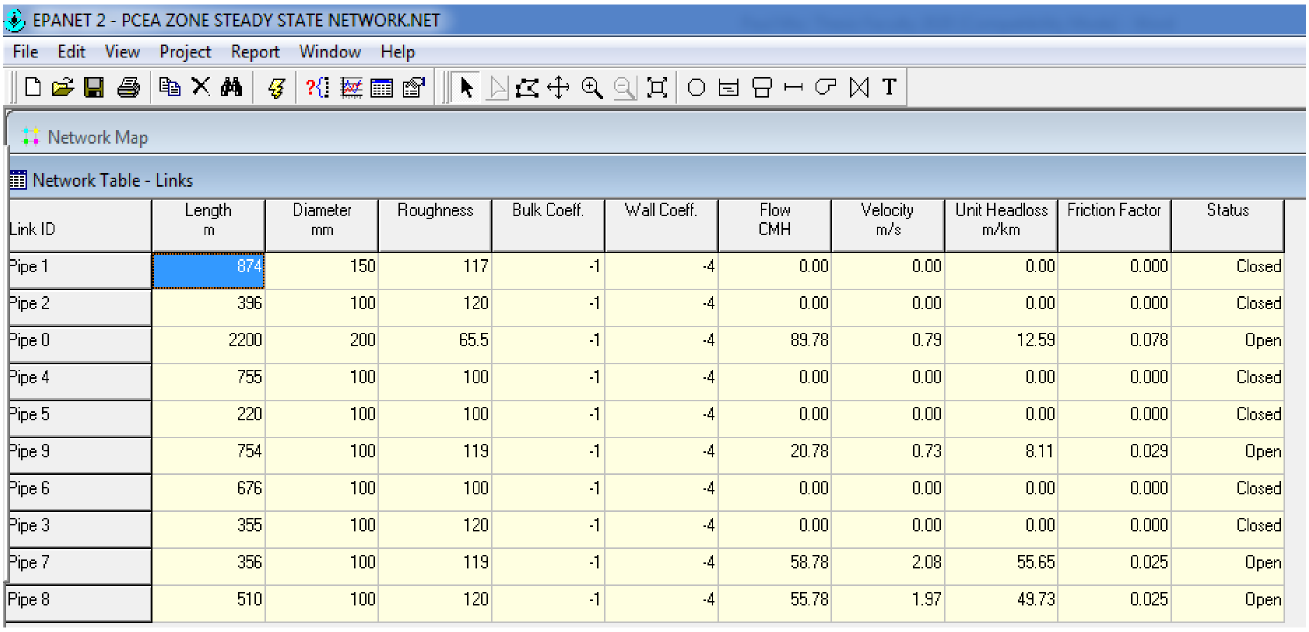

EPANET 2 - PCEA ZONE STEADY STATE NETWORK.NET

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

Fig. 4.1: PCEA Zone Steady State Pipe Network

From Figure 4.1, 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.

4.2.2 Dynamic Simulation

From Fig. 4.2, 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.

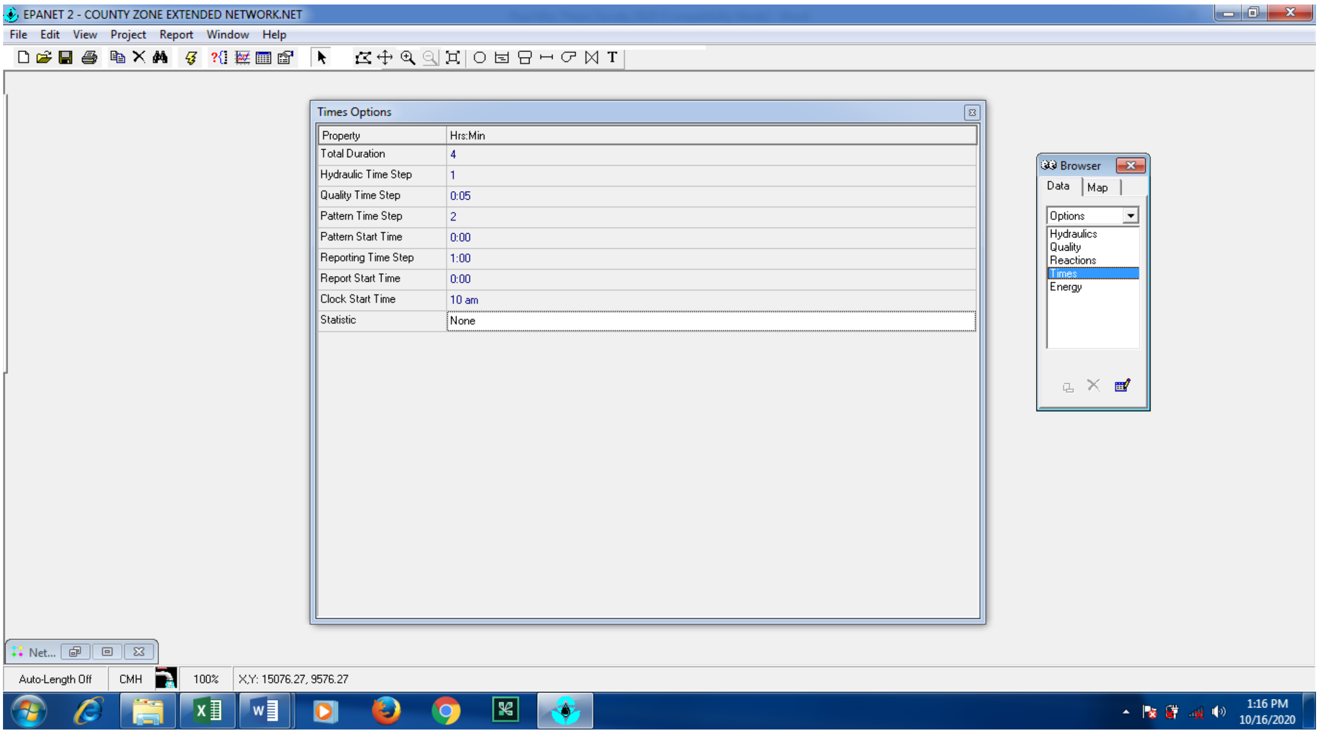

Fig. 4.2: Hydraulic and Pattern Time Step for County Zone Extended Period Network

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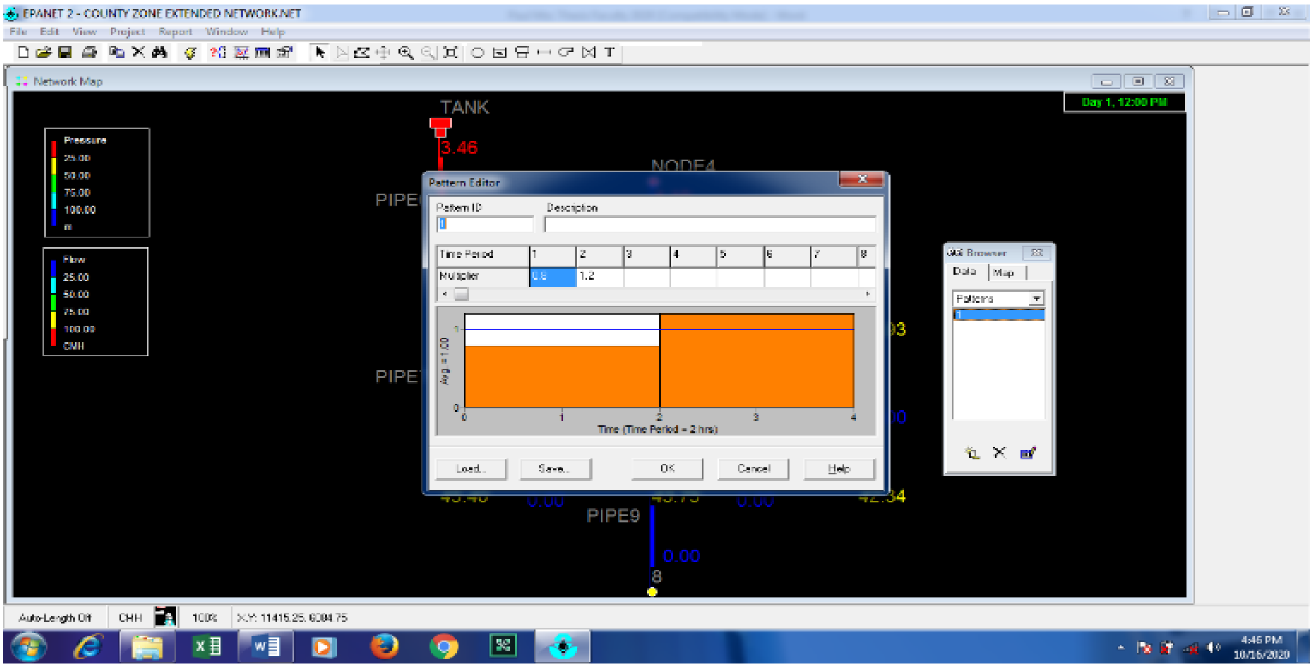

Fig. 4.3: Time Pattern Multipliers for County Zone

Fig. 4.3, 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 of 4 hours duration. During simulations, each pattern Node base demand was multiplied by the respective multiplier. From Fig. 4.4 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 greater than 0.5 are considered acceptable (Santhi *et al.*, 2001).

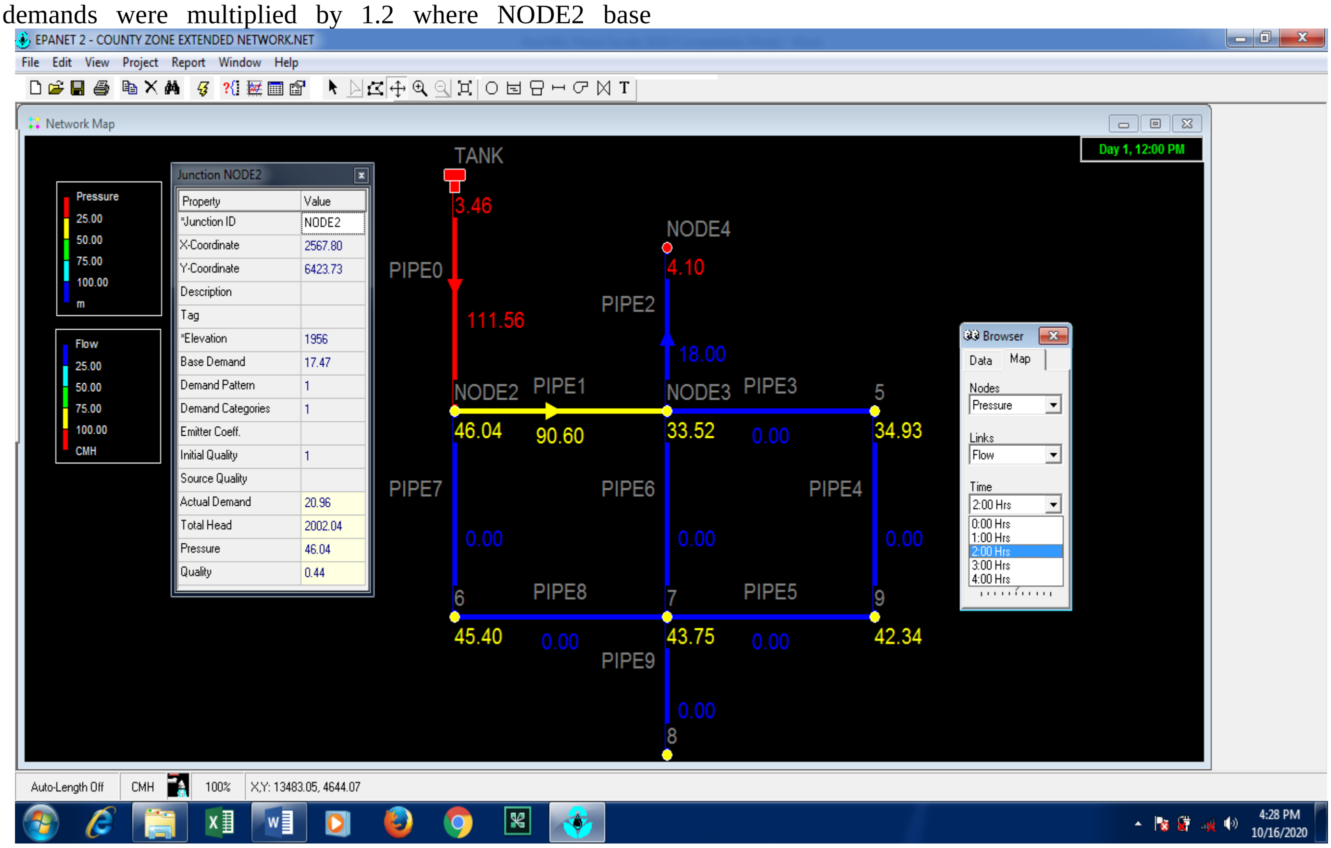

Fig. 4.4: County Zone Extended Period Network Simulated Flow and Pressure (2-hr time step)

PIPE3 are 111.56 m^3/hr, 90.6 m^3/hr and 18 m^3/hr respectively.

4.2.3 Model Calibration

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 4.2.

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Table 4.2: Nash-Sutcliffe Model Efficiency Coefficient (E) for PIPE5 Flow rate (X)

Xobs	Xmodel	Xobs-Xmodel	$(Xobs-Xmodel)^2$	Xobs-28	$(Xobs-28)^2$
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 4.2 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.

4.2.3.1 Measured Versus Simulated Flow And Headloss For County Zone Pipes.

From Fig. 4.5 it is observed that the simulated flow results closely match the measured flow results with coefficients of determination (R^2) of 0.9918. Higher values of R^2 indicate a less error variance and typically values greater than 0.5 are considered acceptable (Santhi *et al.*, 2001).

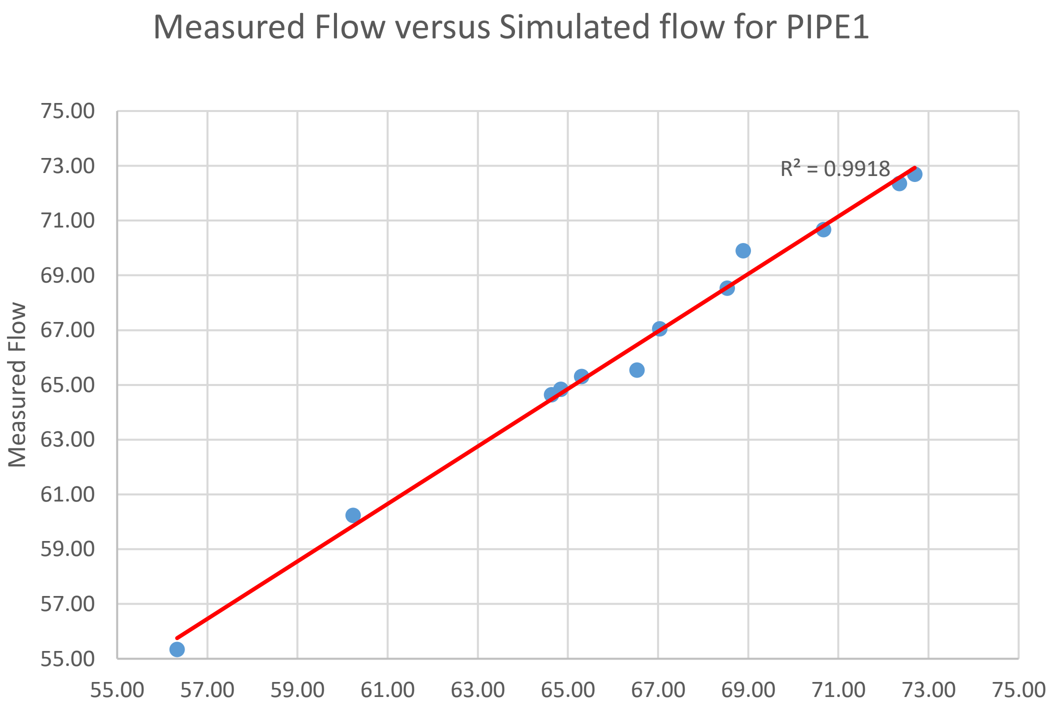

Fig 4.5: Measured Vs. Simulated flow for Pipe 1 in County zone

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Table 4.3: 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 4.3, 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.

4.2.4 Model Validation

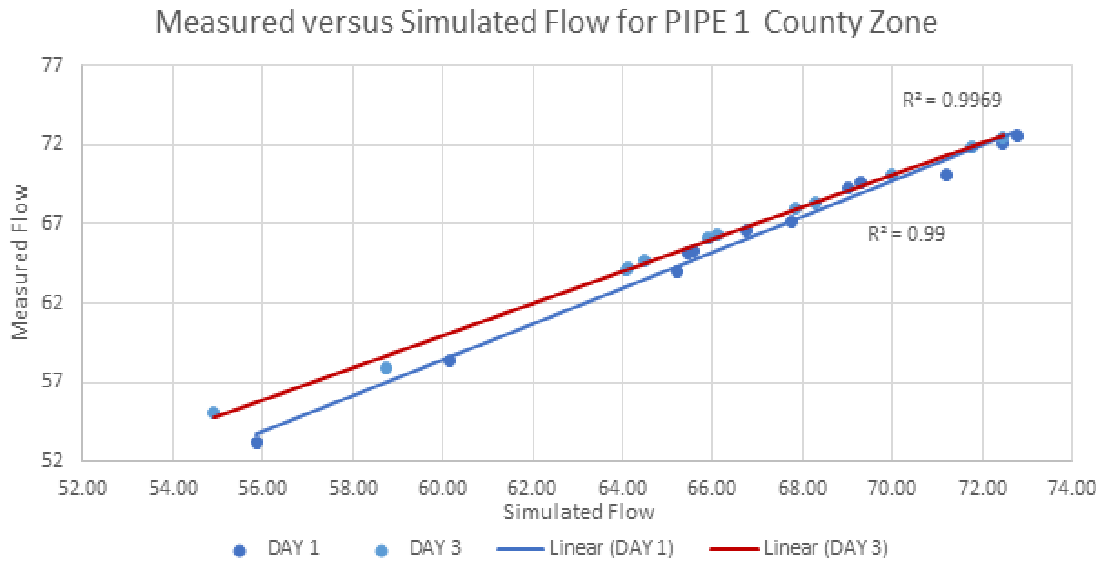

Fig. 4.6: Measured Versus Simulated Flow for Day 1 and Day 2 of PIPE 1 in County Zone

From Fig. 4.6, 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 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 Fig. 4.7.

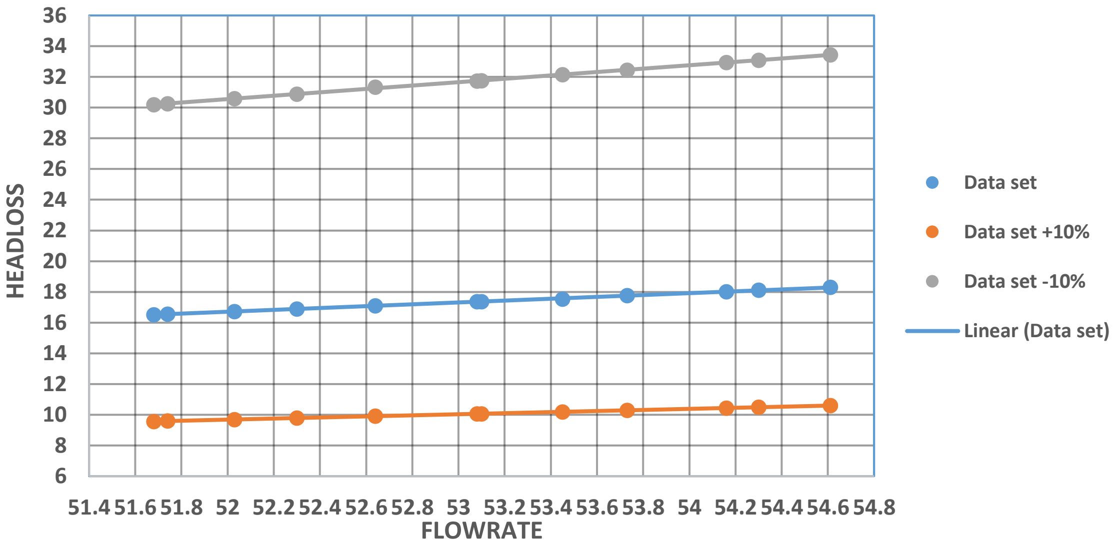

Fig. 4.7: Effect of Varied Independent Variables on Headloss

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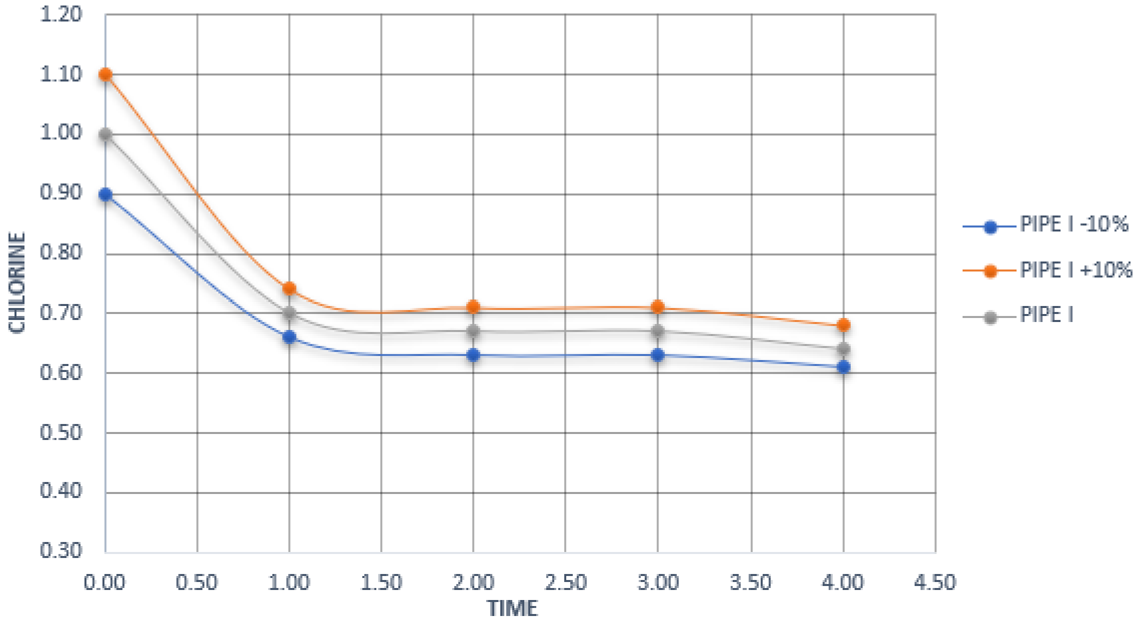

Fig. 4.8: Effect of Varied Independent Variables on Chlorine Decay

Fig. 4.8 shows the head loss output values which were obtained with the +10% and -10% variation of pipe diameter, roughness factor and pipe length. Variation of diameter by +10% resulted to lower headloss values while variation of diameter by -10% resulted to higher headloss values.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. This illustrates a well sensitive model.
From fig. 4.8, 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.

4.3 Impact of Chlorine Decay on The System Hydraulic Performance

4.3.1 Determination of Bulk Reaction Rate K_b

Table 4.4 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 4.4: 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

Fig. 4.9 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.

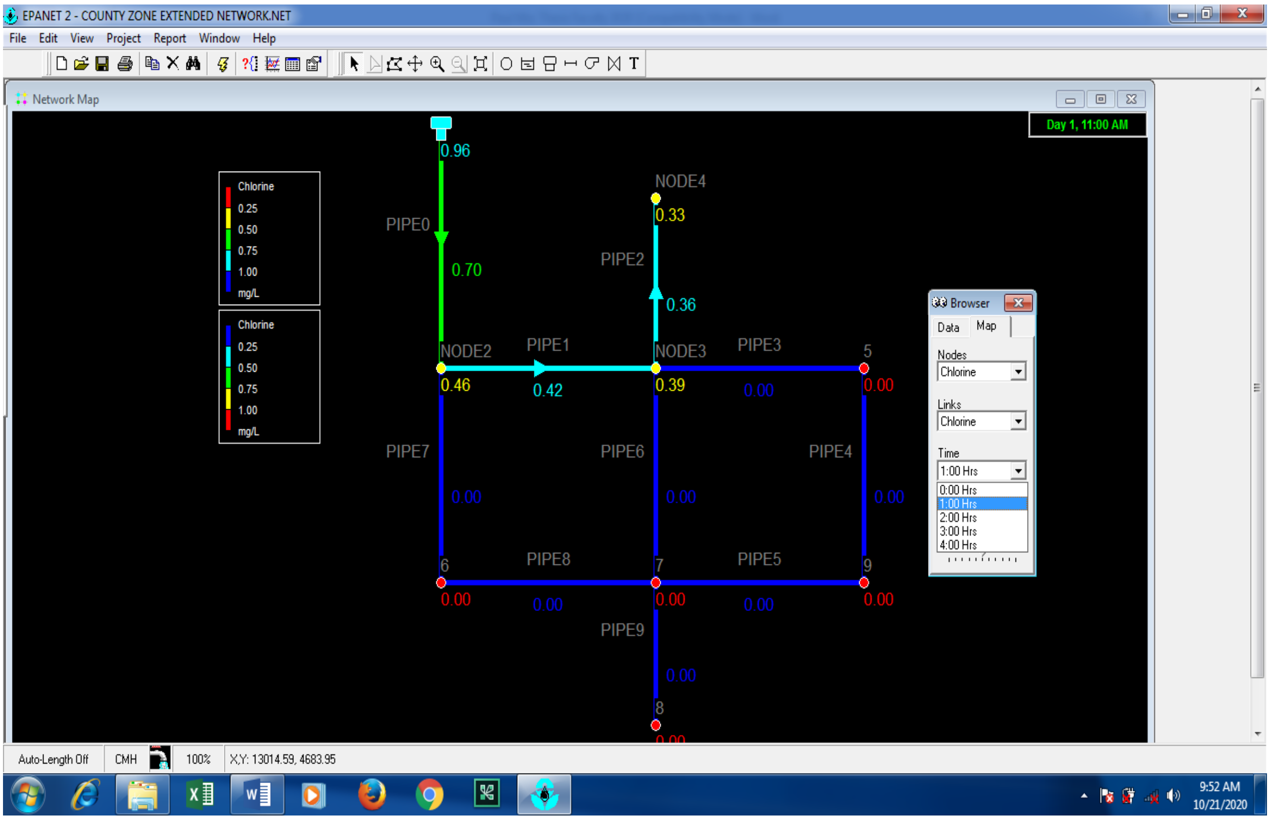

Fig. 4.9: 1.00 Hrs Chlorine Levels in Pipes and Nodes

4.3.3 Chlorine Decay With Respect To Pipe Diameter

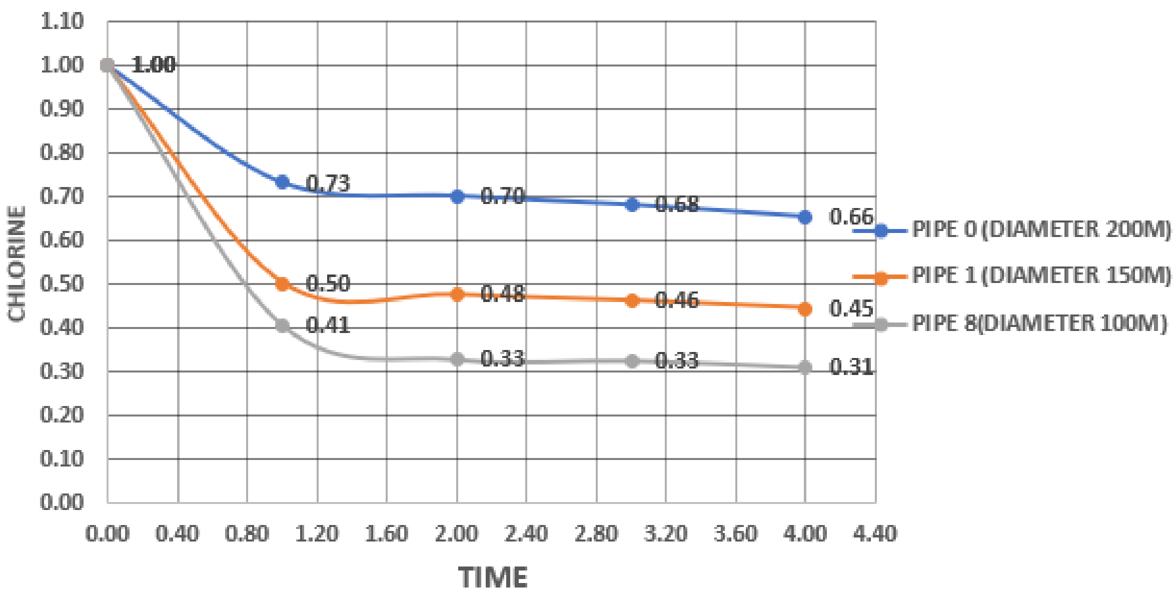

Fig. 4.10: Chlorine Levels vs. Time for Different Pipe Diameters

A graphical presentation of residual chlorine over time with respect to pipe diameter is depicted in Fig. 4.10. 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, 2hrs, 3hrs 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.

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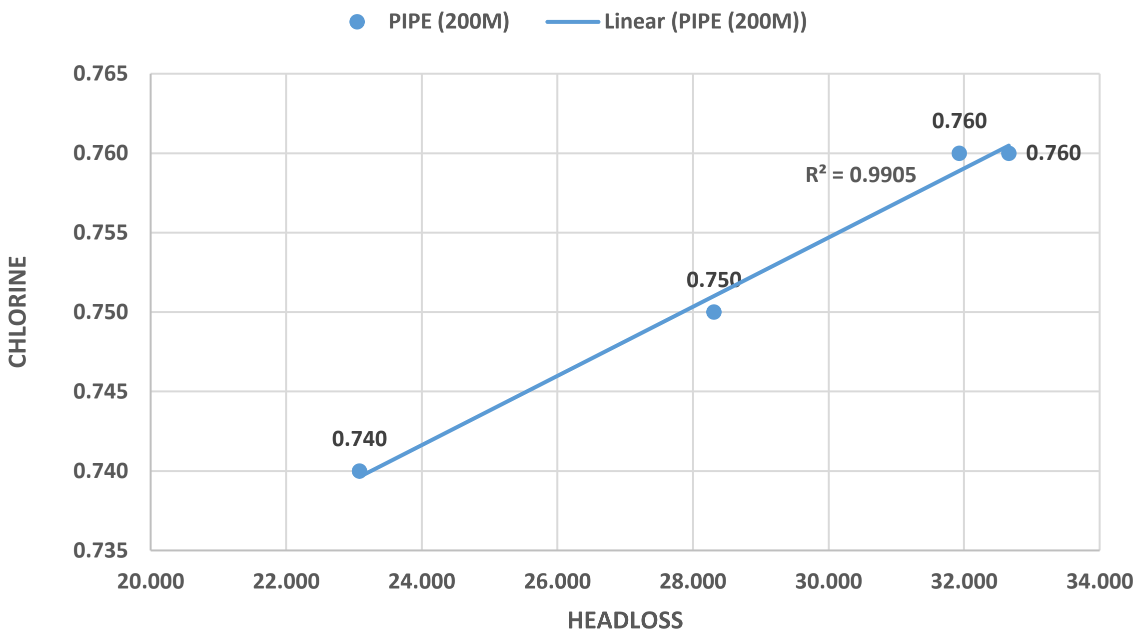

Fig. 4.11: Average Residual Chlorine versus Average Headloss for a 200mm Diameter Pipe

From Fig. 4.11, 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 4.36 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 Fig. 4.11 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.

V. CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

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

1. A regression analysis for pipe headloss as a function of flowrate could be applied to determine the current values for roughness factor of each pipe for the Maralal water distribution network;
2. EPANET model was applied and showed that it could be used as a tool to efficiently simulate and predict at any time the hydraulic performance and water quality for Maralal water distribution system; and
3. 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

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

1. 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;
2. 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
3. Assess the chlorine decay with respect to pipe material and its impact on hydraulic parameters in a non-intermittent/continuous flow water distribution system.

b. Based on the results of this study, Maralal water distribution system should be rehabilitated as follows:

1. PIPE0 (200mm diameter) with the updated roughness factor of 65.5 should be replaced immediately;
2. 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
3. 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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AUTHORS PROFILE

Paul Lolmingani, is a Student pursuing Master of Science in Water Resources and Environmental Management at Egerton University, He obtained a Bachelor of Science degree in Water and Environmental Engineering Egerton University. Registered as a Graduate Engineer by The Engineers Board of Kenya (EBK) under the Engineers Act 2011. Currently he is working as the Chairman of County Public Service Board in the County Government of Samburu. He was a former County Chief Officer Roads, Water and Public in the County Government of Samburu. He worked as Managing Director at Maralal Water and Sanitation Co. Ltd. He also worked at Westgate Community Conservancy as a Project Manager.

Prof. Dr.-Ing. Benedict M. Mutua, is a Full Professor of Water Resources, Hydraulics and Environmental Engineering. He is currently the Deputy Vice-Chancellor (Planning, Partnerships, Research and Innovation) at Kibabii University, Kenya. He obtained his PhD in Water Resources and Environmental Engineering from the Universität für Bodenkultur (BOKU) Wien (University of Natural Resources and Life Sciences, Vienna, Austria. He obtained his MEng. Sc. in Civil and Environmental Engineering from the University of Melbourne, Australia where he specialised in Hydraulics Engineering. He also obtained another MSc. in Water Resources Engineering from the Hebrew University of Jerusalem, Israel where he Specialised in Water Resources and Systems Management). He obtained his BSc. in Agricultural Engineering from Egerton University, Kenya. In addition, Prof. Dr.-Ing. Benedict M. Mutua obtained a Postgraduate Diploma in Water Resources Management from Tel Aviv University, Israel and a Diploma in Soil and Water Engineering from Egerton University, Kenya. He has also done a number of Postdoctoral Research studies in different countries. Prof. Dr.-Ing. Benedict M. Mutua started his teaching and research career in 1990 at Egerton University, Kenya. He has a wealth of experience in teaching, research, administration and Project Management. Prof. Dr.-Ing. Benedict M. Mutua's research interests include and not limited to; Applied Hydraulics, Fluid Mechanics and Dynamics, Water Resources Engineering, Climate Change and Applied Hydrology. He has travelled widely and has collaborative research activities and joint postgraduate students' supervision in several Universities in the world.

Dr. Eng. David N. Kamau, obtained Bsc Agricultural Engineering, at Egerton University, Kenya. He has a Msc. Degree in Water Resources Engineering, University of Dar es salaam, Tanzania. H e attained PhD, in Experimental and Computational Fluid Dynamics, University of Manchester, United Kingdom. Currently He is a Senior Lecturer, Dept. of Civil & Environmental Engineering at Egerton University.