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CASE STUDY

Hydraulic evaluation of water supply and distribution systems: case study of Lawra township water supply system in the Upper-West Region of Ghana

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Abstract

Population growth, urbanization, and people's preferences for water to be brought closest to the home, compel operators with little or no technical know-how on Water supply and distribution systems (WDSs) hydraulics to often introduce new nodes into the network to extend water to consumers. The outcome is operational challenges with some areas not getting adequate supply because of inappropriate pipe routing and sizing, and pressure build up in some sections resulting in pipe bursts during operation. The study evaluated the hydraulic performance of the Lawra township water distribution system for improved service delivery using hydraulic-based models. The Flowius application, Google Earth, Geographical information system (GIS) were the data collection tools employed, while EPAnet and Microsoft Excel software were used to analyse the data. The hydraulic parameters within a 24-hour period in the existing and proposed network were monitored. The existing system runs for a maximum of five hours (00:00am-05:00am) when tested with pressure value (5.3 m) not meeting the minimum pressure requirement of 10 m. The remaining hours of the day depicted negative pressures signifying system unbalance. There were significant head losses in all the three (3) zones (A 24.25 m/km, B 161.66 m/km and C 72.96 m/km) with zone B recording almost thirty-two (32) times higher than the recommended standard of 5 m/km. Series of velocity fluctuation occurred (0m/s – 2.81m/s) across the entire network during the simulation period. The existing network coverage is 70 % with 1360 households. The remodelled network, using EPAnet software for improved performance, was balanced with hydraulic parameters (pressure, velocity and head loss) meeting designed standards and peak hourly demand occurred at 18:00 hours across all the three (3) pressure zones.

Keywords: Water Distribution System, EPAnet, Hydraulic Performance, Peak Hourly Demand

Introduction

One of the most important natural resources available on earth to mankind is water. Water is therefore an essential life-sustaining commodity for all living things, and therefore the responsibility of humanity to preserve it (Abass *et al*., 2022 and Jose and Sumam, 2016). Access to a safe and adequate potable water supply is a basic human right, and water supply systems (WSSs) are the most important public utilities providing safe and potable water to humanity in municipalities across the world (Agunwamba *et al*., 2018). Abbas *et al*. (2022) stress that water is one of the most important natural resources and must be viewed as a means to prosperity and wealth. Public water services provide more than 90 % of water supply in the world (Josea and Sumamb, 2015). Josea and Sumamb (2015*)* further explained that a well-structured conveyance system is a critical component of potable water supply network (WSN) in urban centers (UC) and small towns (ST) and plays a dominant role in the distribution of safe water to consumers.

Water supply and distribution systems (WSDSs) convey water, both in quality and quantity, to our doorsteps. In the past, complex issues on WSDSs were solved using human intelligence. Knowledge in water engineering allowed man to create and transport water from remote places to areas of most need for personal, agricultural, industrial uses, among others. However, Watkins (2016) is of the view that the way water is stored, transported, and distributed may have reached a point beyond which the water supply and distribution systems could begin to create problems than they solve. The hydraulic parameters assessment and performance evaluation of water distribution network (WDN) are therefore necessary. Especially, with the emergence of the United Nation Sustainable Development Goals, specifically target six (6), which seeks to achieve universal and equitable coverage of safely managed drinking water services by 2030, the quality of services water distribution networks provide must be in consonance with acceptable regulations (WHO/UNICEF JMP, 2021).

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Network optimization is often carried out on existing distribution networks to enhance reliability. WDN reliability is the ability of a WDS to meet the demand placed on it, demand reliant on available total volume of water and flow rate, and the pressure range at which the flows must be provided (Trifunovic *et al*., 2015; Torii and Lopez, 2012). Trifunovic´ *et al*. (2015) further stress that WDN optimization process is aimed at incorporating best ways to transport adequate flow of water from source to the customer, taking the constraints into consideration. These constraints could be the minimum and maximum nodal pressure or demand, pipe velocity or unit head -loss, or any other hydraulic parameter that need to be satisfied (Trifunovic *et al*., 2015). The optimal solution to ensure network reliability should therefore consider the best combination of network components (pipes, valves, pumps, tanks, among others) which satisfy the constraints.

In the WDN optimization process, head losses also play a key role in the overall performance of the network. Frictional head losses are mainly due to the fluid viscosity and the flow regime (Haddad, 2019), and their influence may be presented throughout the length of the pipe. The frictional head losses in a long pipe are relatively significant, and so cannot be neglected. Weisbach proposed a mathematical relationship between frictional head losses and pipe length commonly called Darcy-Weisbach equation (Brown, 2002). Typically, the equation links the head loss, the friction coefficient, flow velocity, and the pipe dimensions.

In Sub-Saharan Africa, water infrastructure development remains a challenge, and most WDNs upgrading across Ghana have noticed little or no improvement to match the appropriate hydraulic demand. The Lawra Municipal water supply scheme in Ghana was designed and constructed in the early 1980s and received the first phase of rehabilitation in 1991 under Ghana Accountability for Water, Sanitation and Hygiene (GAWSH) Phase 1 Programme known as GAP which focuses on addressing the gaps and challenges in the water sector. There were twenty-two (22) standpipes situated at vantage locations within the Lawra Township (KNOCKS Consult Limited Technical Report, 2006). There was a second phase of the rehabilitation in 2006 when the number of standpipes reduced to twelve (12) due to an increase in the number of households' connections (KNOCKS Consult Limited Technical Report,

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2006). There has not been any formal consultancy work done on the water system since then to provide data on the hydraulics of the network. However, due to population growth, urbanization, and people's preferences for water to be brought closest to the home, operators with little or no technical know-how on WDSs hydraulics are often compelled to arbitrarily introduce new nodes into the network to extend water to consumers. Introduction of arbitrarily new nodes' usually poses operational challenges with some areas not getting adequate water supply because of inappropriate pipe routing and sizing, and pressure build up in some sections resulting in pipe bursts during operation.

The current water supply network infrastructure in Lawra Township does not meet the present population needs even with the addition of production sources without restructuring or redesigning the system. Since urban life generally features a higher degree of concentrated natural resource consumption including water, WDN must be designed to cater for the ever-increasing water demand (Mu *et al*., 2021). However, the inability of WDS to satisfy demand and pressure requirement is partly due to population increase, largely caused by rapid urbanization (Awe *et al*., 2019). An unimproved water scheme would usually be challenged with unavoidable low- and high-pressure regimes during operation. Poor water systems are also designed because of wrong assumptions, inadequate statistics, and computational input errors, eventually leading to operational difficulties (Omotayo, 2014). These operational difficulties often lead to frequent pipe bursts, loss of treated water often termed non-revenue water (NRW), high repair and maintenance costs, traffic hold ups or diversions and reinstatement of roads before, during and after pipeline repairs. Another side effect of unimproved scheme is back siphonage of dirty and contaminated waters that adversely affects water quality delivered to consumers (Omotayo, 2014).

The research main objective is to evaluate the hydraulic performance and network coverage and structure of the Lawra township WDS for improved service delivery using hydraulic-based models. Based on this objective, the study would model the existing network to evaluate the impact of arbitrary new nodes on hydraulic parameters in the network, model a proposed network to correct hydraulic parameters if any and evaluate the existing network structure and coverage.

Methods and Materials

Study area

Lawra is one of the important towns in the Upper-West Region of Ghana opportune to have a WDS. It was elevated to Municipal Assembly (MA) status on 15^{th} March 2018, and geographically situated between latitude $10^0$38'27.68''N and longitude $2^0$53'25.85" W (Google Earth Lawra, Ghana, 2022). The township has an area of 6.17 km^2 with an average slope and elevation of 0.0012 and 170 m respectively (Google Earth Lawra, Ghana, 2022). Lawra is located at 85 km from the Regional Capital, Wa, and the built-up area of the town is combined to the West and most agricultural lands (Google Map Lawra, Ghana, 2022). Figure 1 shows the map of the study area.

The Black Volta is about 2.3 km away from Lawra Township and provides water to the inhabitants of the town for dry season farming through a small-scale irrigation system. Though Lawra community is predominantly Christian, a significant number of Muslims, and a few Traditionalists live there. The main water supply source is groundwater, usually pumped using four (4) submersible pumps – two (2) 4 kw and two (2) 2.2 kw capacities through a 30 m^3/h package treatment plant into a 120 m^3 concrete high-level tank (HLT) for distribution through gravity.

Data collection tools

Goggle Earth and Flowius application were the data collection tools used for this study. While the Google Earth application helped to carve out the study area, the Flowius application assisted in picking spot heights or elevations. Detailed use of these two (2) applications are discussed below.

Google Earth application

A computer application called Google Earth reads three-dimensional maps of the planet, mostly from satellite images. The application allows users to view cities and landscapes from different perspectives by mapping the Earth by

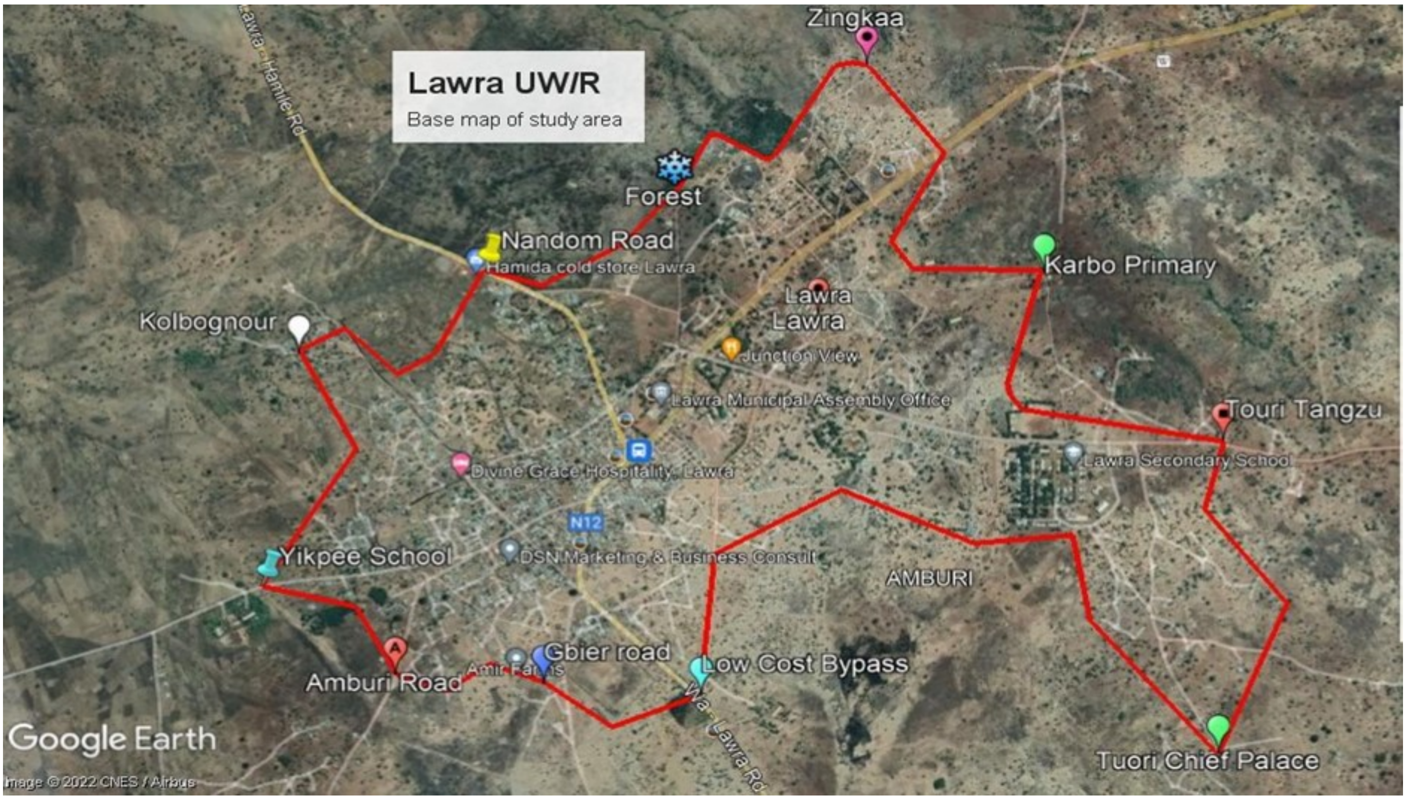

Figure 1 Google Earth study area map (Google Earth, 2022)

superimposing satellite photos, aerial photography, and GIS data onto a three-dimensional globe. With a keyboard or mouse, users can use addresses and locations to explore the world. You can also download the program on a tablet or smartphone and navigate with a pen or touch screen. With Keyhole Markup Language, users can add more data to the software and submit it to different websites, such blogs or forums. Google Earth is a tool that allows you to overlay different types of photographs on the earth's surface.

Flowius application

Flowius is a free productivity app developed by Flowius, and its latest version (Flowius1.0) was released on 07/09/2016 and updated on 26/09/2020. The Flowius application was used to map out the boundaries of the study area for the pipe network. All spot heights or elevations, length between nodes of the existing pipe network was mapped which was imported into pipeline design software (PDS) for the purpose of this study, EPAnet software for performance evaluation of hydraulic parameters. Flowius app was equally used to map out infrastructure network such as roads, water sources, distribution points, bridges, and all that needed investigation. The mapping was achieved by walking to the point of interest, add the point, and the Flowius app collected all the required data. Flowius simplifies the process of mapping communities, it was easy and natural to use, and it eliminated the burden of surveying teams working for days with minimal training.

Data analysis tool

The main tool used in the data analysis was the EPAnet software. The EPAnet software was used to model the existing and proposed WDN for hydraulic parameters performance. The input data used covered the various pipes sizes of the distribution network, the elevations of the various nodes in the existing network leading to obtaining the total pipe length and network configuration of the existing system.

The study employed the extended period simulation to assess the realistic nature of the network and the hydraulic parameters over a day period as demand varies. The existing and proposed networks were classified into three (3) zones to determine areas with similar hydraulic parameters characteristics. Understanding of the hydraulic parameters' characteristics will inform the best operational strategies for better network performance. In the existing network, three (3) pipes (links) from each of the three (3) zones were selected for hydraulic parameters (unit head loss, velocity and flow) analysis. In a similar way, three nodes from each of the three (3) zones of the existing network was selected to observe the pressure variation at demand nodes in the three (3) difference zones. The reason for selecting different /same pipes diameters in the same zones were to reduces biasness since pipes diameters play crucial role in the kind of flow, velocity or head loss that exist in the links. The selection of the different/same pipe diameters in the same zone also give a fair understanding of hydraulic parameters behaviours at the different sections of the network. The same approached of pipe diameter and nodes selection was applied in the proposed network for hydraulic parameters analysis. The study however was not comparing the two networks but rather correcting the anomalies that would be found in the existing network if any to better develop a proposed network that will be robust enough to accommodate the required hydraulic demands.

A well-designed network must run successfully if tested with all hydraulic parameters meeting designed standards. As the EPANET tool is very user-friendly, the following procedures were adopted with the spot heights/elevations collected using the Flowius application and imported into the EPAnet software for further data analysis:

- Imported the distribution network points into Epanet tool from Flowius app;
- Connected all the points with links to complete the layout of the network;
- Fixed the total demand from reservoir or tank based on the hydraulic scheme design;
- Assigned the units of flow in liters per second (LPS), and fixed the head loss formula to Hazen-Williams (H – W) method with roughness factor of 140 for plastic polyvinyl chloride pipes (uPVC);
- The hydraulic properties of pipes such as diameter, roughness, among other, were designed variables obtained during simulation;
- Thoroughly checked that pipes and nodes were properly connected at intersections and reservoir nodes; and
- Run the hydraulic analysis.

Population and demand calculation

Lawra township has a population of 12,500 and the per capita demand, and daily peak factor used were 100lcd and 1.2 respectively (National Population and Housing Census, 2020). Five percent (5 %) of the water supply was added as fire demand. For reservoir capacity, 35 % - 40 % of the daily average consumption was used, and a residual pressure head of 10 meters (Small Town Sector Guidelines, 2010). The population was projected over ten (10)-year design period using 2022 as the base year, the year the research was conducted. An annual population growth rate 2.5 % – 3.5 % of district or the region was used to project the future population (Ghana Statistical Service, 2020). It is practically impossible to eliminate unaccounted for water (UFW) or non-revenue water (NRW) in a WDS, especially in developing countries where WDSs perform at sub-optimal levels. Therefore, an acceptable level of 15 % UFW was used, and the losses mostly included flushing (Small Town Sector Guidelines, 2010).

Demand forecasting for the town was projected for a period of ten (10) years beginning from 2022 with an average regional growth rate (r) of 2.5 % – 3.5 % for the projected population. The water demand was estimated using the projected population calculated with Equation (1).

$$P_n = P_0\left(1 + \frac{r}{100}\right)^n \qquad (1)$$

Where P_n = projected or future population after n decades, P_o = based/initial population, r =average growth and n = the number of decades. The average growth rate (r) could be computed from available population data using Equation (2).

$$r = \left(\frac{P_2}{P_1}\right)^{\frac{1}{t}} - 1 \qquad (2)$$

Where P_1 = initial known population, P_2 = final known population, t = number of decades between P_1 and P_2.

The growth rate values were computed for each known decade, the average taken as the assumed constant per decade increase (r). The average could again be the arithmetic average or the geometric average. Using an average regional growth rate (2.5-3.5 %) and an average peak daily water demand of 1.2, the projected demand is calculated for a ten year period to be 16001.

Results and Discussion

Hydraulic model of the existing and proposed water distribution system

The existing Lawra Township WDN was modelled to evaluate its hydraulic integrity, which included the network ability to meet demand placed upon it with provision for adequate pressure, fire protection, and reliable uninterrupted water supply. This section provides the results and discussion, leading to key findings of the study. The network hydraulic parameters, namely unit head loss per km, pressures at demand nodes, velocities, and flow rate in pipes were discussed.

Table 1 Current water demand estimation

Current population p	Per capita demand (lpcd) a	Domestic demand (m^3/d) b=p*a	Five (5%) fire demand (m^3/d) c=b*1.05	Fifteen (15%) water losses (m^3/d) d=c*1.15	Fifteen (15%) commercial dater demand (m^3/d) e=d*1.15	Peak daily demand (1.2) (m^3/d) f=e*1.2
12,500	100	1,250	1,312.5	1,509.38	1735.79	2082.94

Source: Ghana Statistical Service Office, Lawra (2020); CWSA revised guidelines (Design Guidelines, 2021).

Table 2 Projected water demand estimation

Current population p	Per capita demand (lpcd) a	Domestic demand (m^3/d) b=p*a	Five (5%) fire demand (m^3/d) c=b*1.05	Fifteen (15%) water losses (m^3/d) d=c*1.15	Fifteen (15%) Commercial dater demand (m^3/d) e=d*1.15	Peak daily demand (1.2) (m^3/d) f=e*1.2
16001	100	1,600.1	1680.11	1932.13	2221.95	2666.34

Source: Ghana Statistical Service Office, Lawra (2020); CWSA revised guidelines (Design Guidelines, 2021)

Hydraulic parameters performance in zone A of the existing WDN

Pressure distribution of some selected nodes in zone A of the existing system

Figure 3 below illustrates the pressure distribution of three (3) selected nodes in zone A. Observation from the pressure values indicate that between 00:00am to 05:00am, the network run successfully with pressure values ranging between 34.62m to 56.44m at all the three (3) selected nodes. However, after 05:00am, the system run into negative pressures at all the selected nodes. Which then means that the network losses its hydraulic integrity after 05:00am. Indication of negative pressures suggest that there are pipes breaks/leakages, which will eventually results in water loss and pressure drop in the pipelines. Pressures drop in the system can lead to back siphonage of contaminants into network due to atmospheric pressure been higher than that of the system pressures because of the system pressure drop. Another indication of the negative pressure is that the pipe diameters are smaller to deliver the required demand at the nodes.

Velocity distribution of some selected links at zone A of the existing network

There were series of low velocity values (0m/s-0.03) especially in link 147 and link 45 occurring at different time during the simulation period. It is difficult to identify any trend in the velocity distribution in Figure 4. Link 50 portrays a haphazard nature of velocity distribution, recording 0 m/s as the lowest

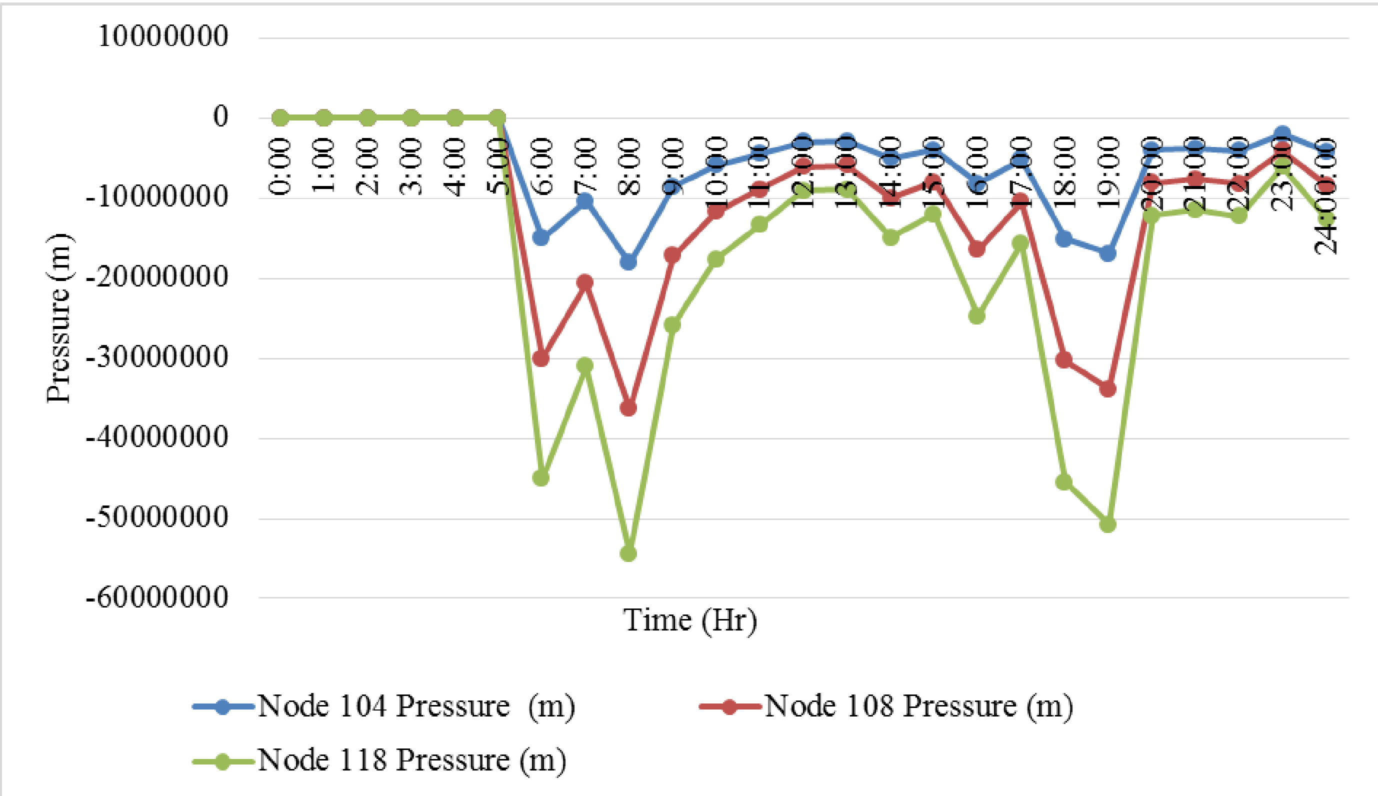

Figure 3 Pressure distribution at zone A of the existing system

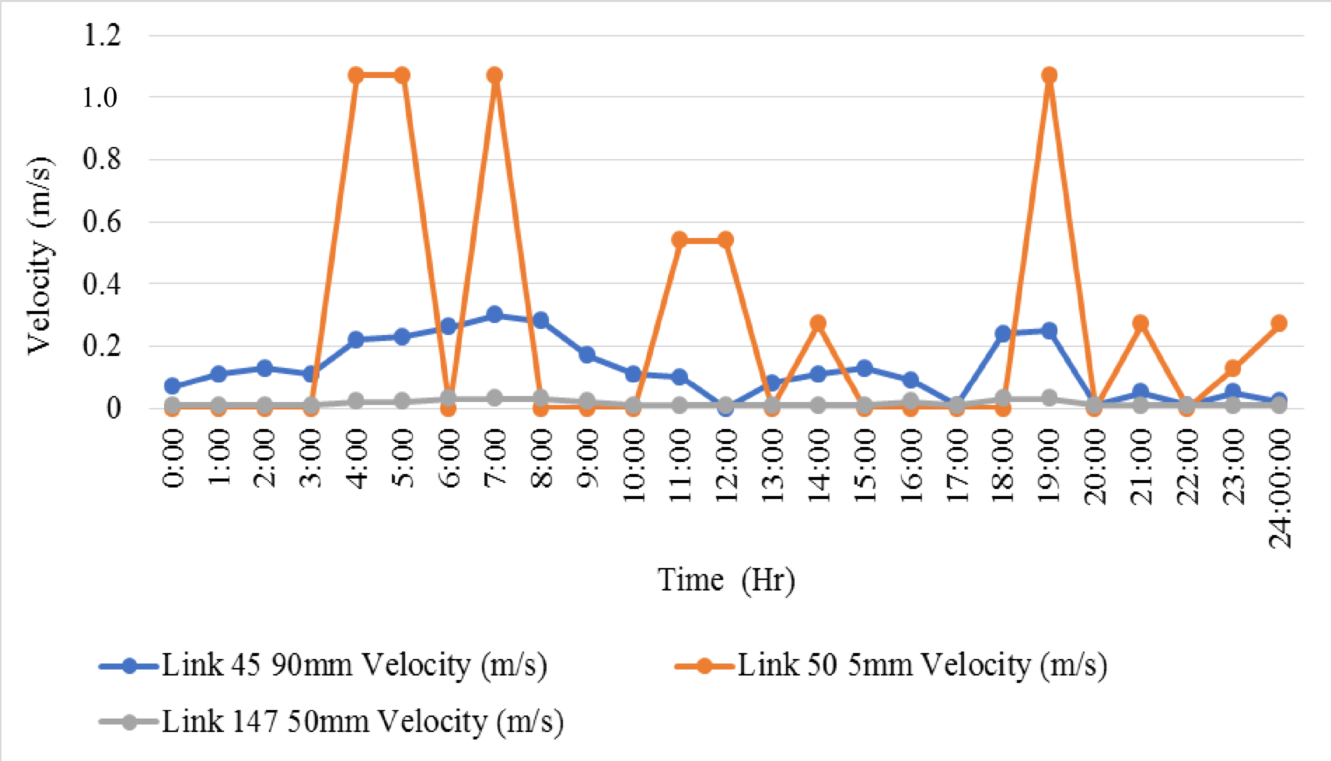

Figure 4 Velocity distribution of selected links in zone A of the existing system

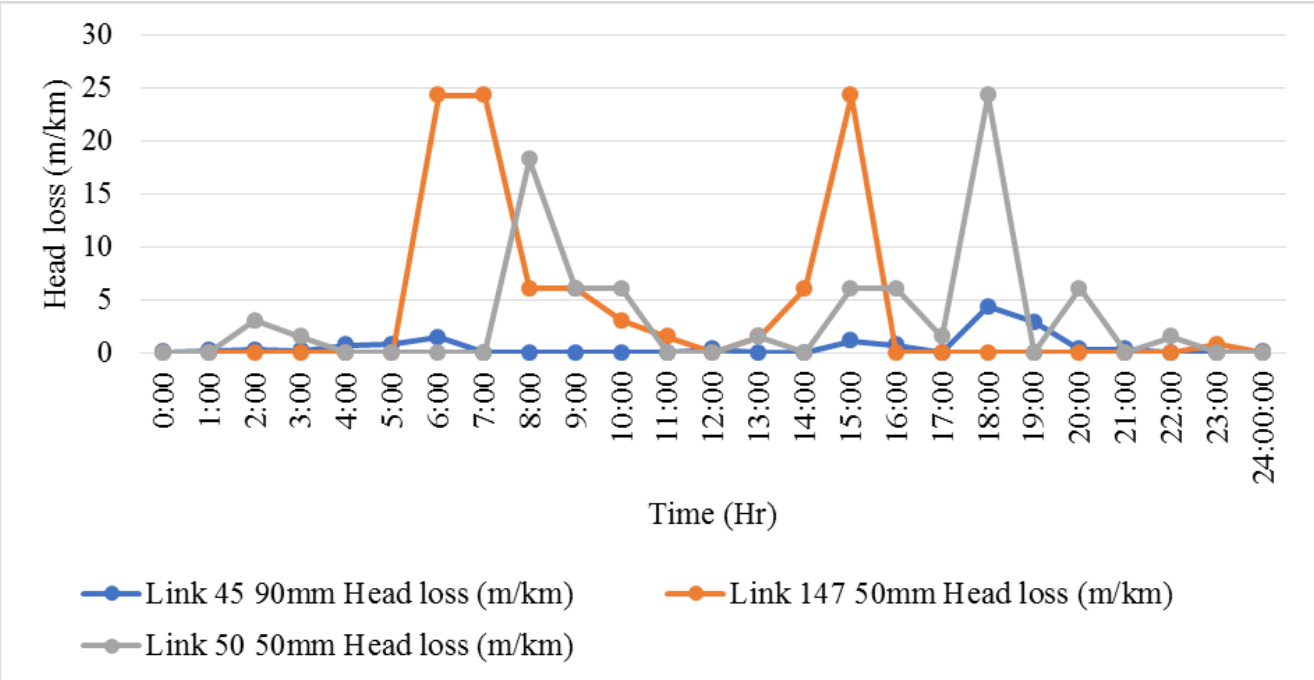

Figure 5 Head loss distribution of selected links in zone A of the existing system.

and 1.07 m/s as the highest velocity. These lower velocities in zone A can lead to sediments deposition and biofilm formation which have dire consequences on water quality.

Head loss distribution of some selected links at zone A of the existing network

Figures 5 depict the unit head losses (m/km) distribution in the zone A of the existing network. In the Community Water and Sanitation Agency (CWSA) guidelines on water systems design, a maximum of 5m/km is the recommended standard. Head losses greater than this value (5m/km) do not guarantee safety and efficient performance of the network. The results of the head loss distribution are very high above the standard which generally depicted flow resistance or friction in pipes.

Hydraulic parameter performance in zone B of the existing WDN

Pressure distribution of selected nodes in zone B of the existing system

Figure 6 shows a line graph of three (3) nodes selected from Zone B to observe the pressure variation at the different nodes in the same zone. When the network was tested, it runs successful from 00:00 am to 5:00am in node 140 and 00:00am to 03:00am in node 67 and node 103 respectively. The positive pressure values are between 19.5m to 40.99m. The remaining hours of the day recorded negative pressures which reflect the network inability to supply the required demand to consumers.

Velocity distribution of selected links in zone B of the existing network

The velocity profile of the selected links in zone B of the

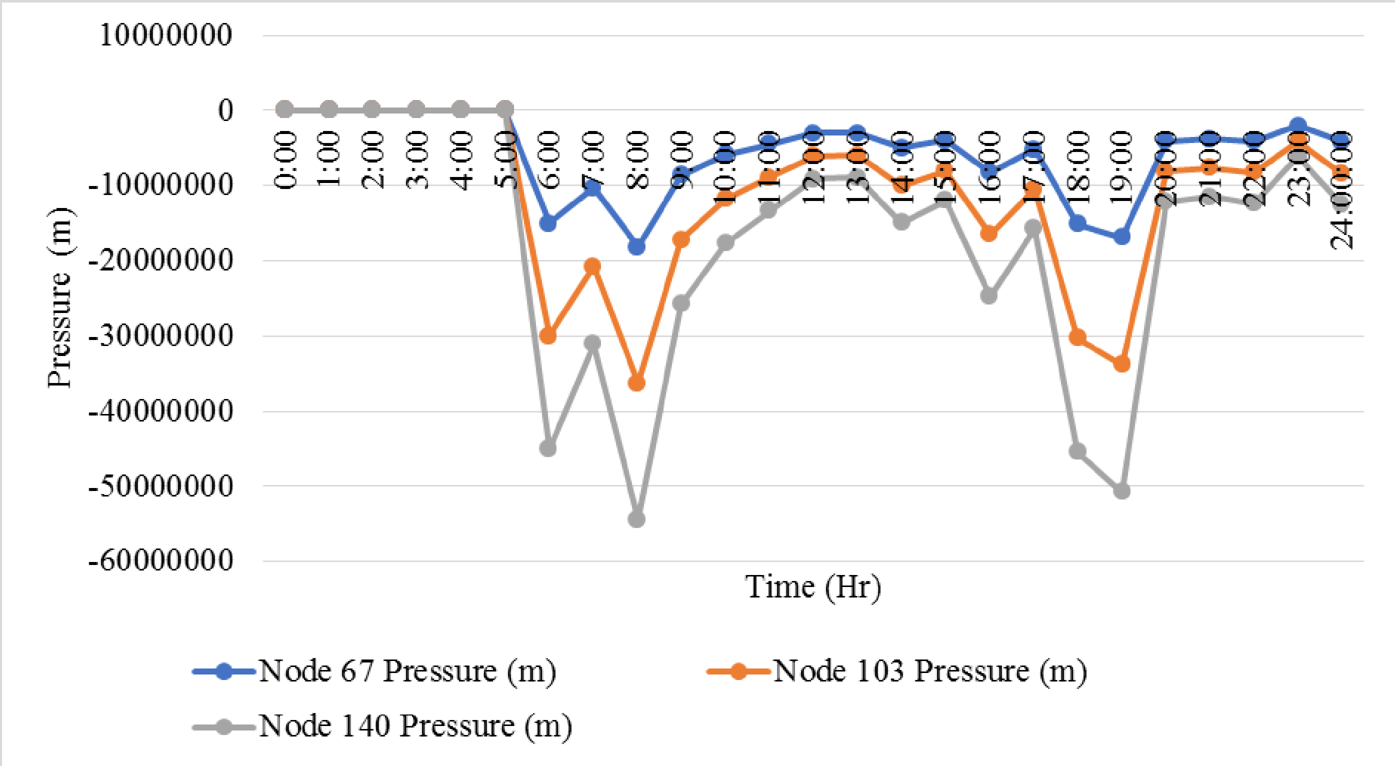

Figure 6 Pressure distribution of selected nodes in zone B of the existing system

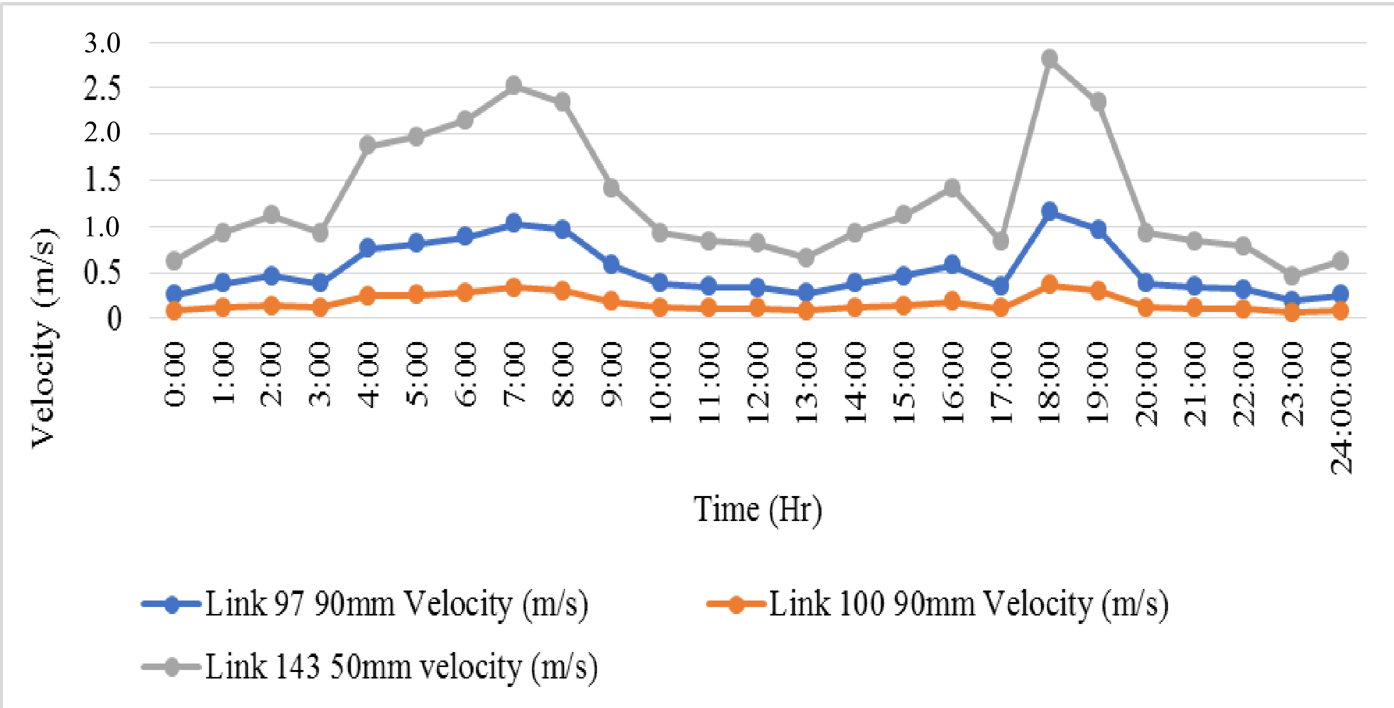

Figure 7 Velocity distribution of selected links in zone B of the existing network

existing system depict a recognizable trend with time. Even though some relative low velocities were observed in link 100 in Figure, but there was a trend. In the same zone B, higher velocities ranging from 2m/s to 2.81m/s occurred in link 143. Some observed trends were also identified. In all the three (3) links, it was observed that between 04:00am - 08:00am and 18:00pm -17:00pm, there was an increase in velocities at these hours. The increase in velocities suggest that there is also an increase in withdrawal of water from the network. This withdrawal can be consumption caused by consumers or leakages caused by pipes breaks.

Head loss distribution of some selected links at zone B of the existing network

In the Community Water and Sanitation Agency (CWSA) guidelines on water systems design, a maximum of 5 m/km is the recommended standard. Head losses greater than this value (5 m/km) do not guarantee safety and efficient performance of the network. The highest unit head loss (161.8 m/km) occurred in link143 with diameter of 50mm and of length 550m. This is almost thirty-two (32) times higher than the recommended head loss value per kilometre. It was also found in figure 8 above that head losses were inversely proportional to the pipe diameters, which meant that the smaller the pipe size, the higher the head loss values. The graph also depicts an increase in head losses between 03:00am -09:00am and 17:00pm - 20:00pm in all the links in zone B. The higher head loss values at these hours are signs of higher demand with an increase in flow causing frictional losses

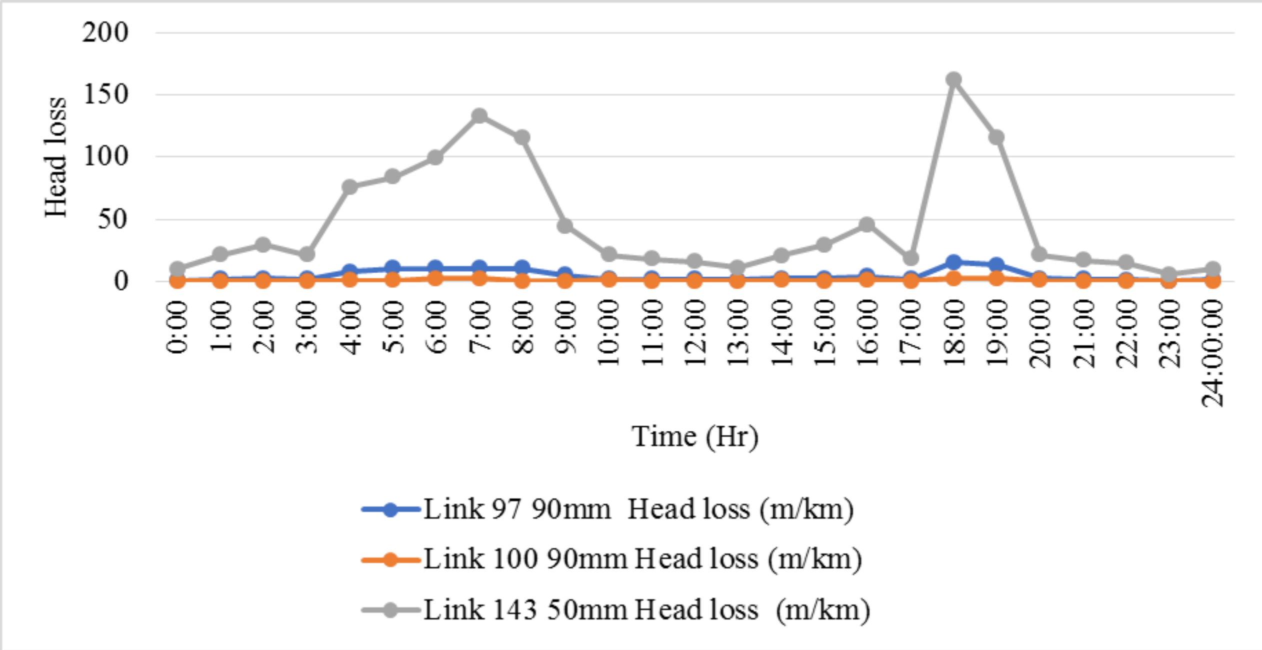

Figure 8 Head loss distribution in zone B of the existing system

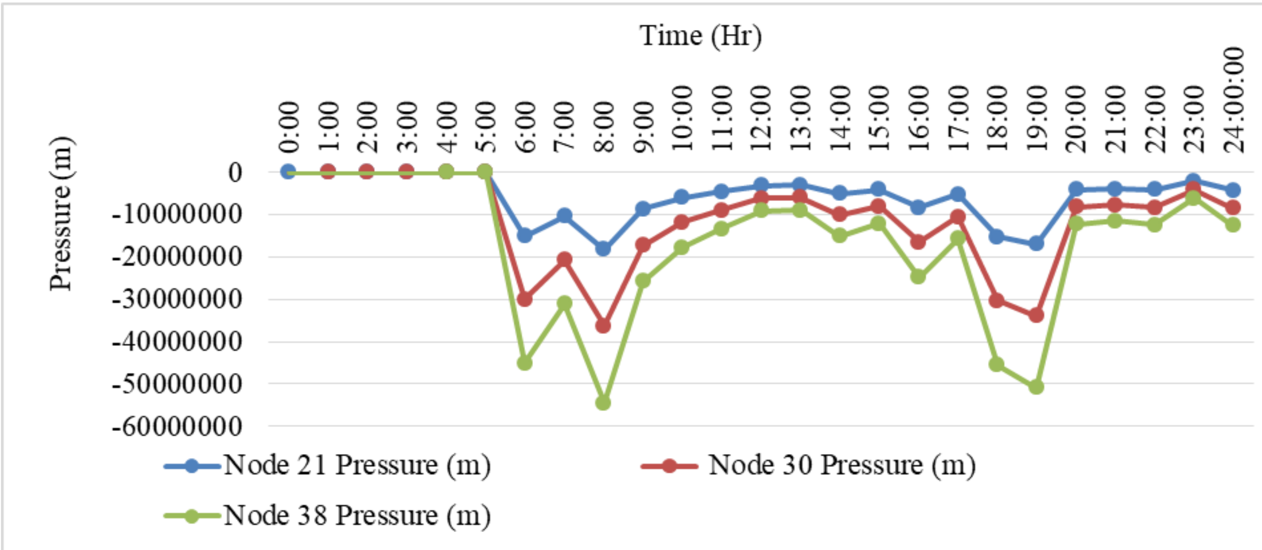

Figure 9 Pressure distribution of selected nodes in zone C of the existing system

Hydraulic parameters performance in zone C of the existing network

Pressure distribution of selected nodes in zone C of the existing system

Figure 9 in zone C shows that the pressure distribution graphs followed a similar trend as depicted in Figure 6 of zone B. When the network was tested, in zone C, it runs successful from 00:00 am to 5:00am in node 21 and node 38 respectively and 00:00am to 03:00am in and node 30. The positive pressure values are between 5.3m to 30.68m. The remaining hours of the day recorded negative pressures which reflect the network inability to supply the required demand to consumers. In this zone, there were a series of pressure values below the minimum pressure head of 10m as stipulated in CWSA guidelines

Velocity distribution of selected links in zone C of the existing network

The velocity profile of the selected links in zone C of the existing system in figure 10 below did not vary much as compare with what is depicted in zone B in Figure 7. A recognizable trend of velocity values with time was observed. The velocity values in all the three (3) links were considerable good with lowest velocity value been 0.1m/s and highest value of 1.83m/s. It was also observed that between 04:00am - 08:00am and 18:00pm -17:00pm, there is an increase in velocities at these hours. The increase in velocities suggest that there is also an increase in withdrawal of water from the network. This withdrawal can be consumption caused by consumers or leakages caused by pipes breaks.

Head loss distribution of some selected links at zone C of the existing network

Head losses in zone C were comparatively high in link 89 as against the other two links (4 and 5) even though all three links are 50mm in size. There was a relatively lower head at the node connecting link 89 (319.99m) as compared to 334.89m and 335.08m connecting link 5 and 4 respectively. The lower head at the node connecting link 89 has an advantage of receiving more flow, which eventually will cause frictional head loss in the pipe.

General hydraulic parameters performance of the existing network

The evaluation of the hydraulic performance of the existing system proved neither effective nor efficient because of the kind of level of services it is providing. The network only runs

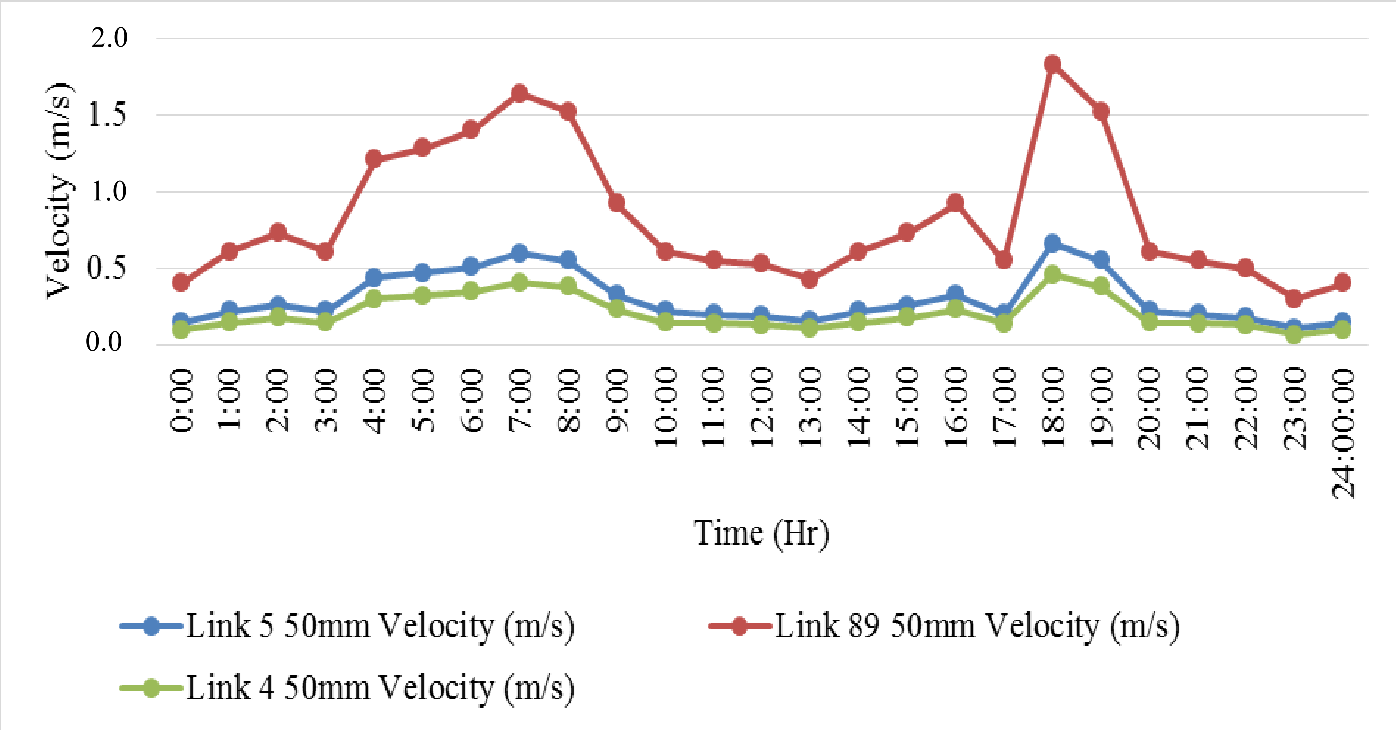

Figure 10 Velocity distribution of selected links in zone C of the existing system

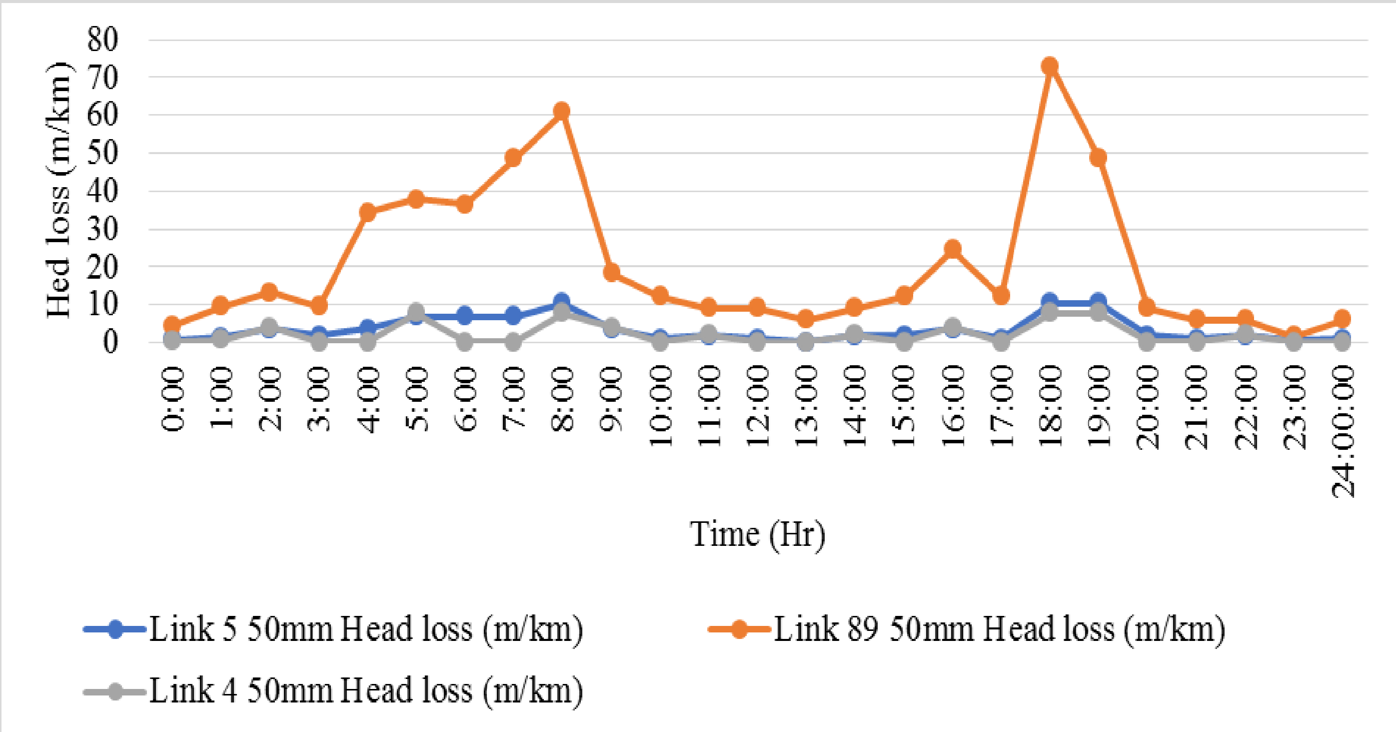

Figure 11 Head loss distribution of selected links in zone C of the existing system

for a maximum of five hours (00:00am -05:00am) when tested with pressure values ranging from 5.3m-56.44m and the remaining hours of the day depicting negative pressures which are signs of system unbalance. In some sections (zone C) of the network, the 5hrs run could not even maintain the 10m minimum pressure as stipulated in CWSA guidelines for pressure requirements. There were series of velocity fluctuation across the entire network with some links recording 0m/s at some hours within the simulation period. This has water quality implications as lower velocities are good grounds for sediment settlement and biofilm formation. Lower velocities are also signals of low flow in links which will lead to demand not been met or satisfied. This problem of low velocities can be resolved by adjusting pipe diameter to improve velocities while monitoring other hydraulic parameters performance. Head loss in the network was generally bad with all the three (3) zones (A 24.25m/km, B 161.66m/km and C 72.96m/km) recording higher head losses in some of the links. Head loss values exceeding 5m/km are generally not acceptable in designs as recorded in CWSA guidelines on water system design. These higher losses are manifestation of operational challenges such as pressure drop, leakages, increase pumping cost, reduction in water supply leading to customer agitation and mistrust. The hydraulic integrity of the existing network is therefore not robust and need remodelling to meet current and future demand.

Hydraulic parameters performance of the proposed network

Pressure distribution of selected nodes in zone A of the proposed system

From Figure 12 in zone A, A similar pressure distribution pattern was observed at all the three (3) selected nodes where the pipes lines were charged between 00:00 hours and 4:00 hours, indicating less withdrawal of water from the network at such hours. There was also an observed fairly uniform pressure distribution between 4:00 hours and 9:00 hours, signifying less demand fluctuation. However, demand suddenly increased at 18:00 which led to a drop in pressure at that hour. The pressure drop occurred in all the three nodes in the zone (A) indicating the impact of high demand across the entire zone A. The peak hourly demand in the zone occurred at 18:00 pm. The pressure values in Figure 12 are within the minimum and maximum pressures of 10m and 60m respectively as stated in the guidelines of smalltown water design by CWSA.

Velocity distribution of selected links in zone A of the proposed system

Figure 13 displayed the velocity profile of the proposed network in zone A with a recognized trend of velocities in the different links in zone A of the network. According to the continuity equation (Q=VA). Discharge/flow rate is directly proportional to velocity which means that an increase in Q will lead to an increase in V and the reverse is the same. The highest velocity observed at 18:00pm indicates that there was higher demand or higher flow rates in the links at that hour causing higher volume of water withdrawal from the system. The velocity profile trend also showed that demands were relatively higher increasing velocity in the links between 04:00am and 10:00am. This also means that flow rates were equally higher at such hours (04:00am-10:00am) to meet the desire demand. The velocity values in zone A could generate self-cleansing velocities to prevent deposition of debris/ sediment in the pipelines.

Head loss distribution of selected links in zone A of the proposed network

In zone A of Figure 14, head loss distribution in the selected links showed a similar trend as velocities in figure 13. Where

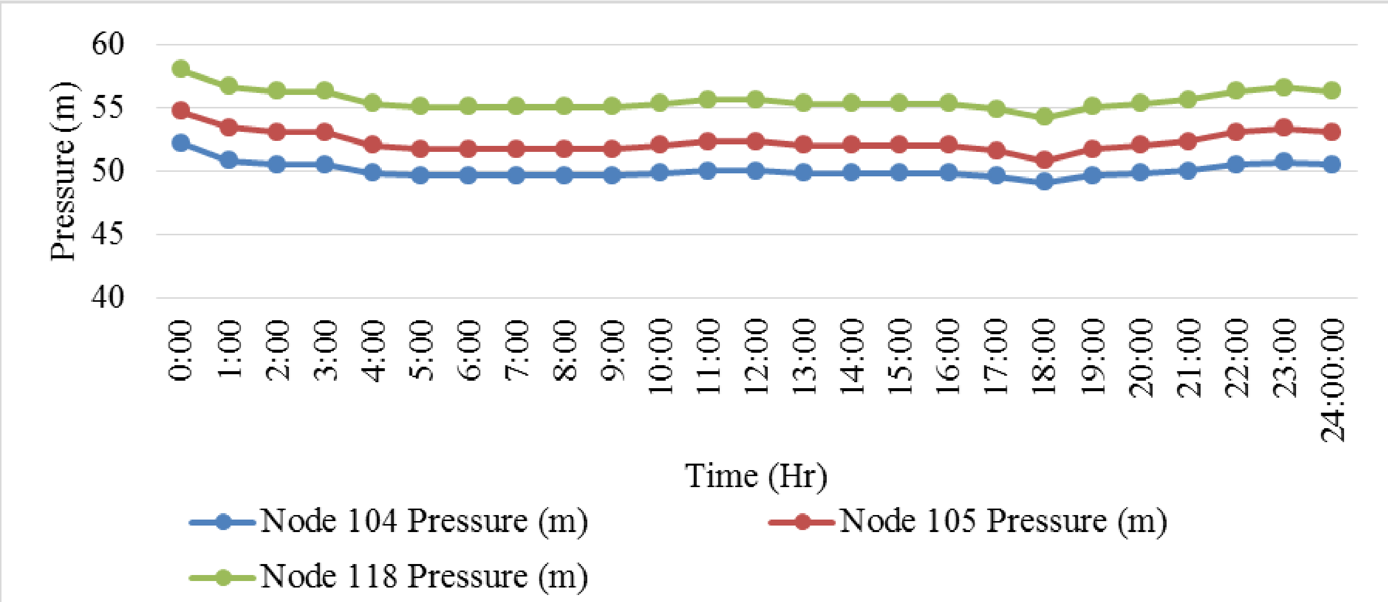

Figure 12 Pressure distribution of selected nodes in zone A of the proposed system

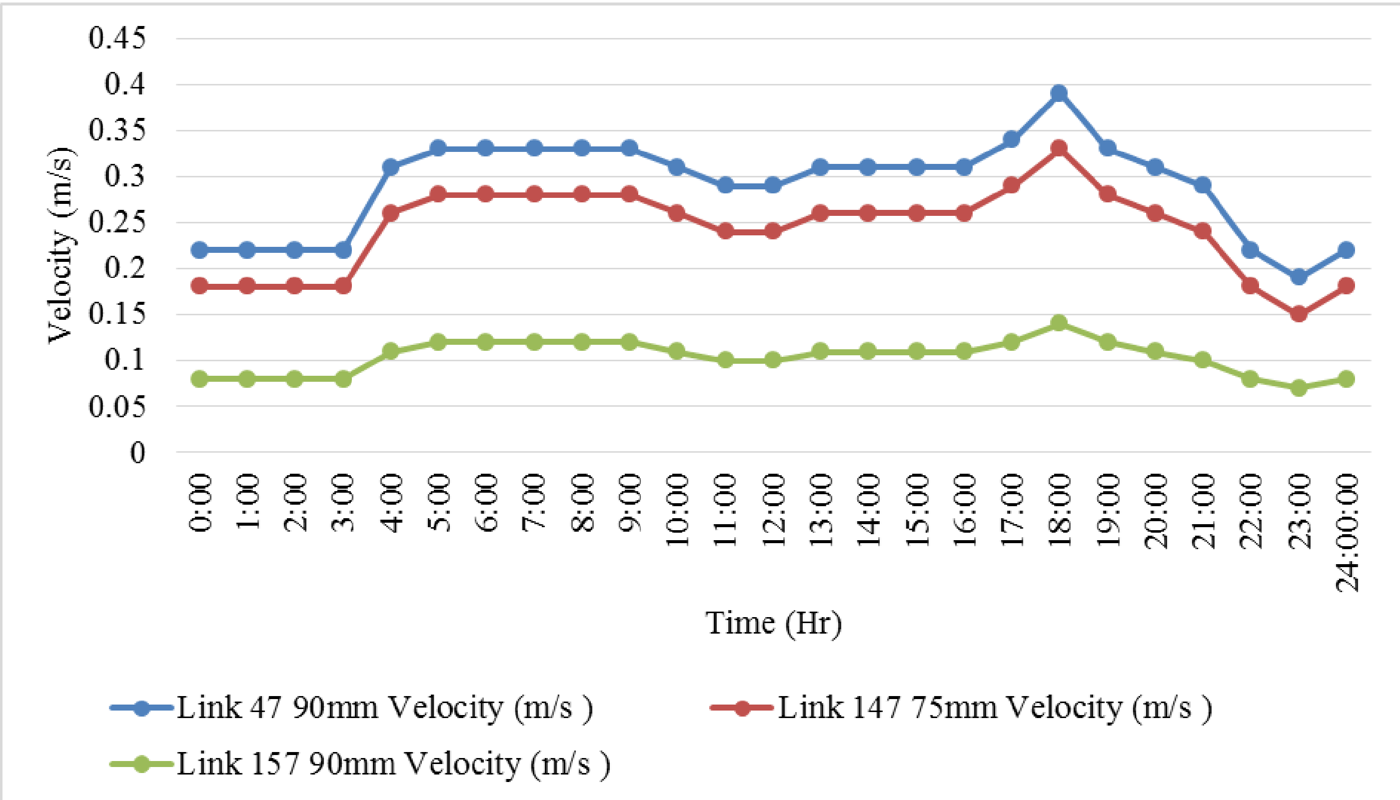

Figure 13 Velocity distribution of selected links in zone A of the proposed system

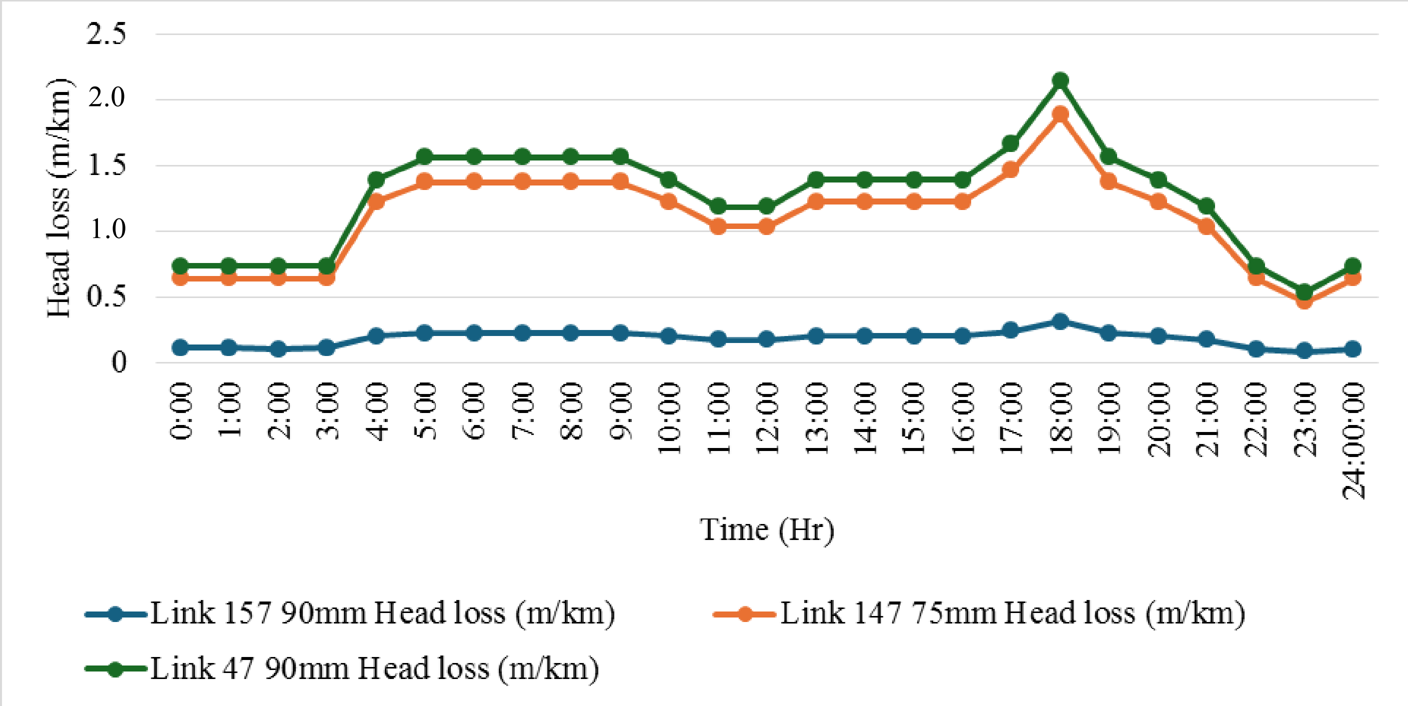

Figure 14 Head loss distribution of selected links in zone A of the proposed system

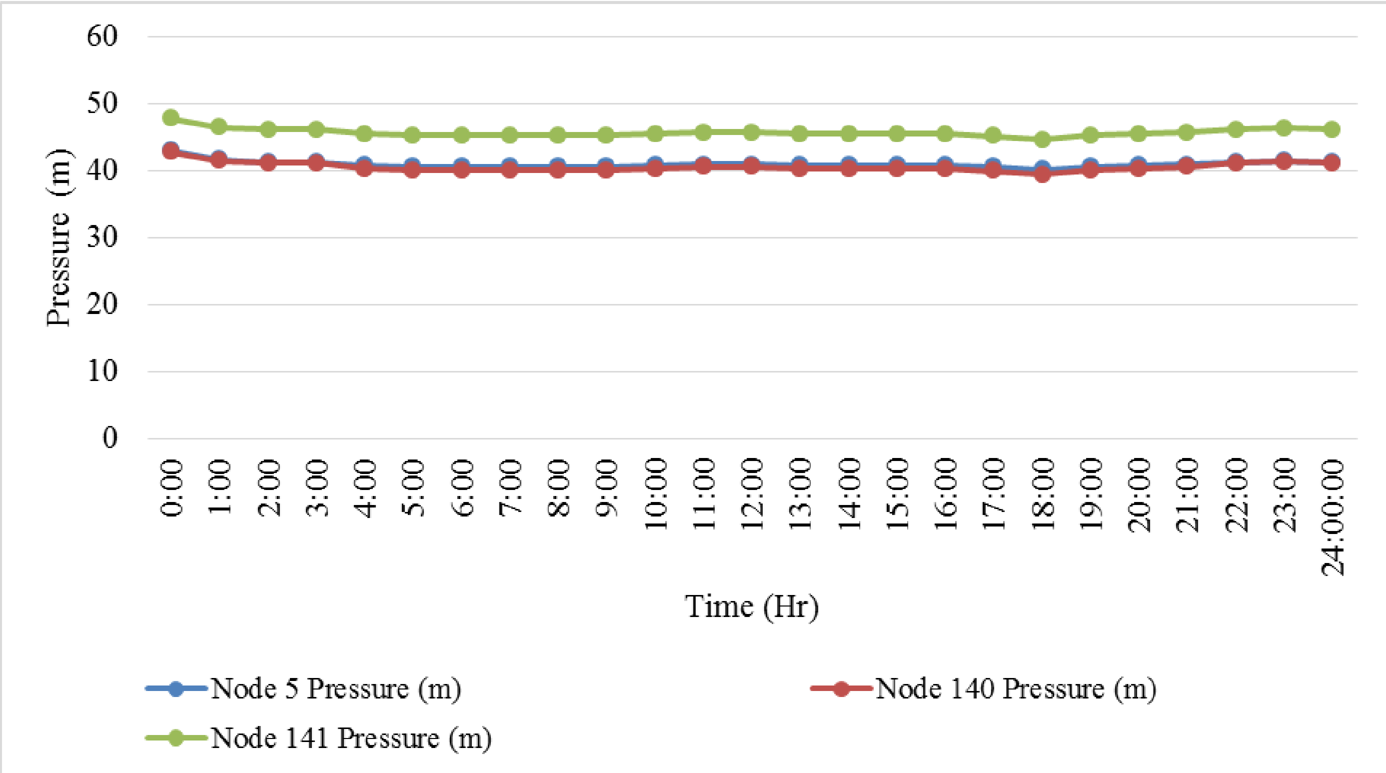

Figure 15 Pressure distribution of selected nodes in zone B of the proposed system

there was in increase in velocity values, there was also an increase in head loss values at that same period. Higher head loss values was observed at 18:00pm in Figure 14, likewise velocity values at the same time frame. The interesting thing was that the higher head loss value (2.14m/km) in the zone (link 47) of figure 14 was still within CWSA guidelines of a maximum of 5m/km. The head loss characteristics in figure 14 headloss formulae. An increase in velocity or decrease in diameter lead to an increase is in frictional headloss as the flow force through a narrow diameter. Head losses above this value (5m/km) does not guarantee safety and efficient performance of the network.

Pressure distribution of selected nodes in zone B of the proposed network

The pressure distribution trend in zone B of figure 15 has no significant difference from that of zone A in figure 12. The only observe difference is the pressure ranges. In figure 15, the pressure values range between 39.37m to 47.75m which are still within the acceptable pressure level of 10m minimum pressure and 60m maximum pressure

Velocity distribution of selected links in zone B of the proposed network

The velocity distribution in Figure 16 are fairly distributed depicting good system performance in terms of velocity profile of the system velocity. Issues regarding low and high velocities are not associated with any of the links in this zone.

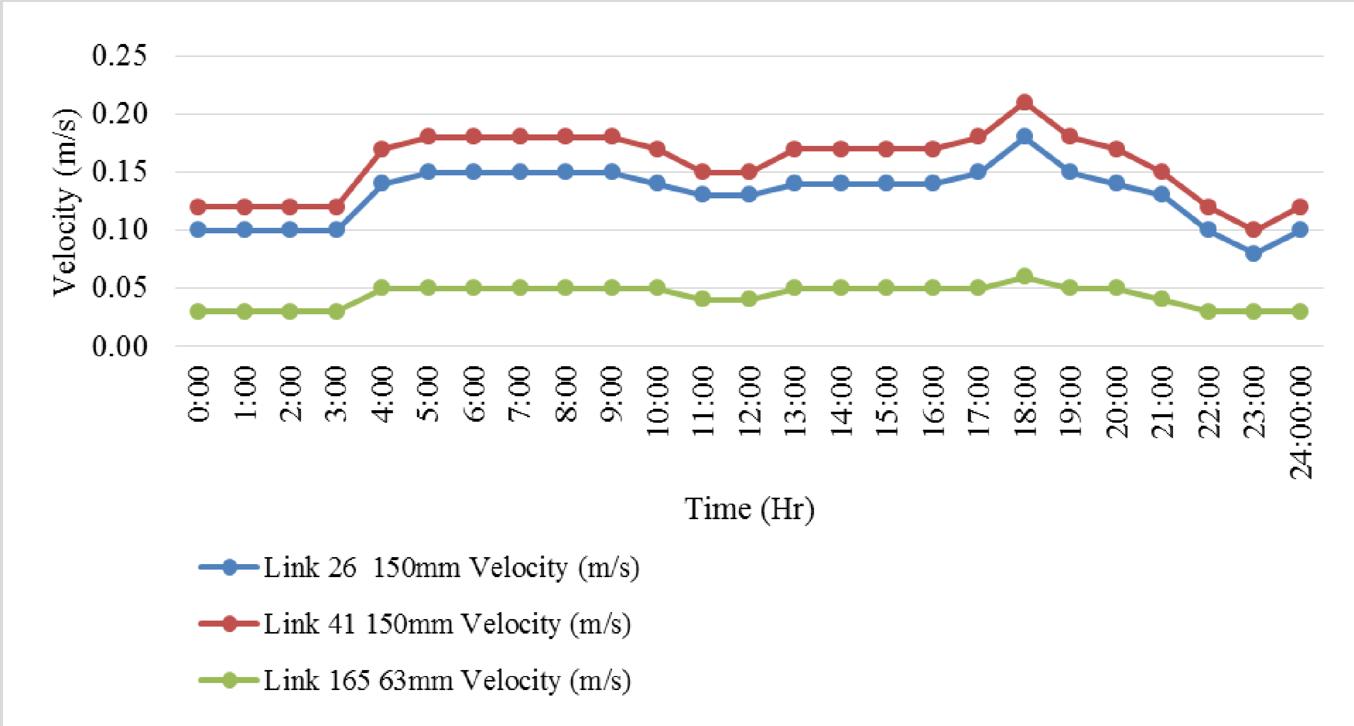

Figure 16 Velocity distribution of selected links in zone B of the proposed system

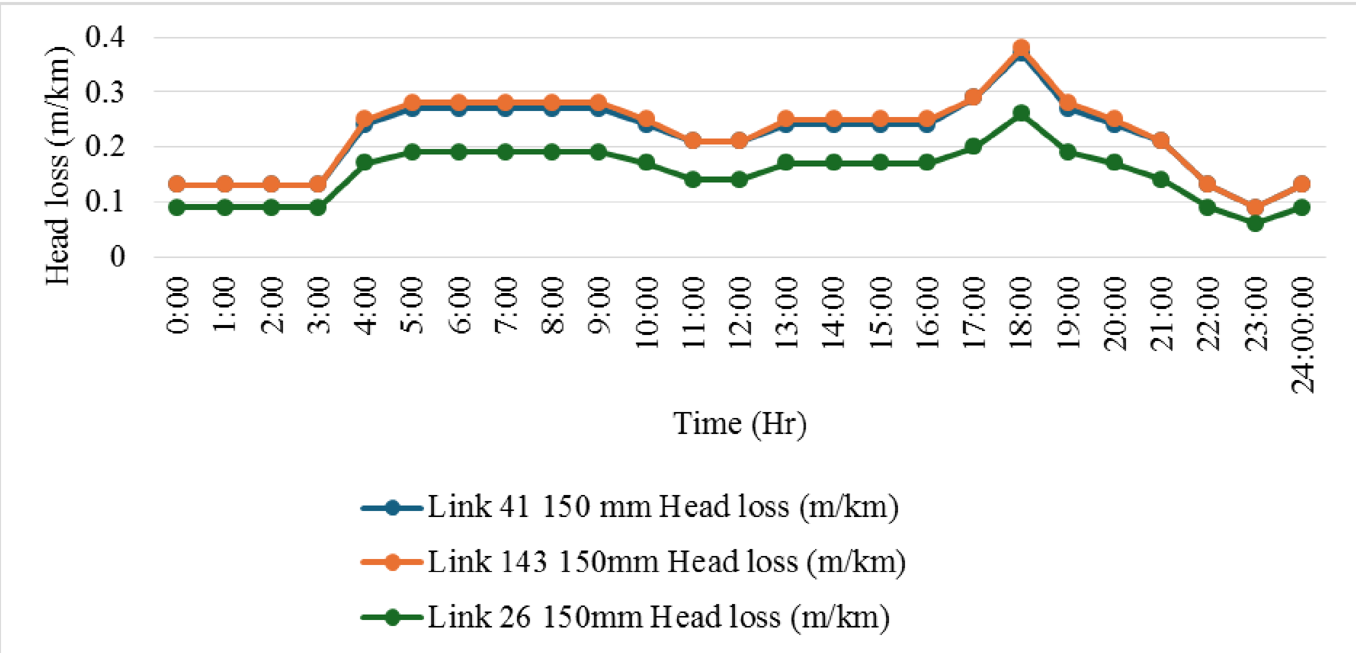

Figure 17 Head loss distribution of selected links in zone B of the proposed system

Head loss distribution of selected links in zone B of the proposed network

Generally, the profile of head loss values in Figure 17 are within design standards. At the early hours of the day (00:00am -03:00am) all the selected links showed very low headloss values. This could be associated with less demand from consumers leading to less flow rates and less interaction of inner pipe wall with the flow, thereby producing insignificant frictional forces. Even though demand increase within the day, frictional losses were still minimal.

Pressure distribution of selected nodes in zone C of the proposed network

Observing from Figure 12, 15 and 18 of zone A, B and C respectively of the proposed system, the pressure distribution followed a similar trend of showing higher pressures in the early hour of the day and dropping at 18:00pm. All though lower pressure values (14.76m-33.04m) occurred in zone C, but the pressure distribution trend is the same across all the three (3) zones. Zone C has the highest elevations in the network topology which are the signs of the low pressure values. The pressure requirement in this zone still met the minimum pressure standard of 10m.

Velocity distribution of selected links in zone C of the proposed system

Figure 19 depicts the velocity characteristics in the selected links in zone C of the proposed system. These links represent the general behaviour of velocity in the zone which the pattern take the same trend as displayed in figure 16 of zone B above. There was a relatively constant velocities in all three (3) links between 00:00am -03:00am which presupposed that flow rates were also constant at these hours (00:00-03:00am). Velocities began increasing as consumers withdraw water from the network. However, the velocity values never went below 0.05m/s or increase above 0.33m/s. The range of velocity values in the selected links are good to maintain the desired flows.

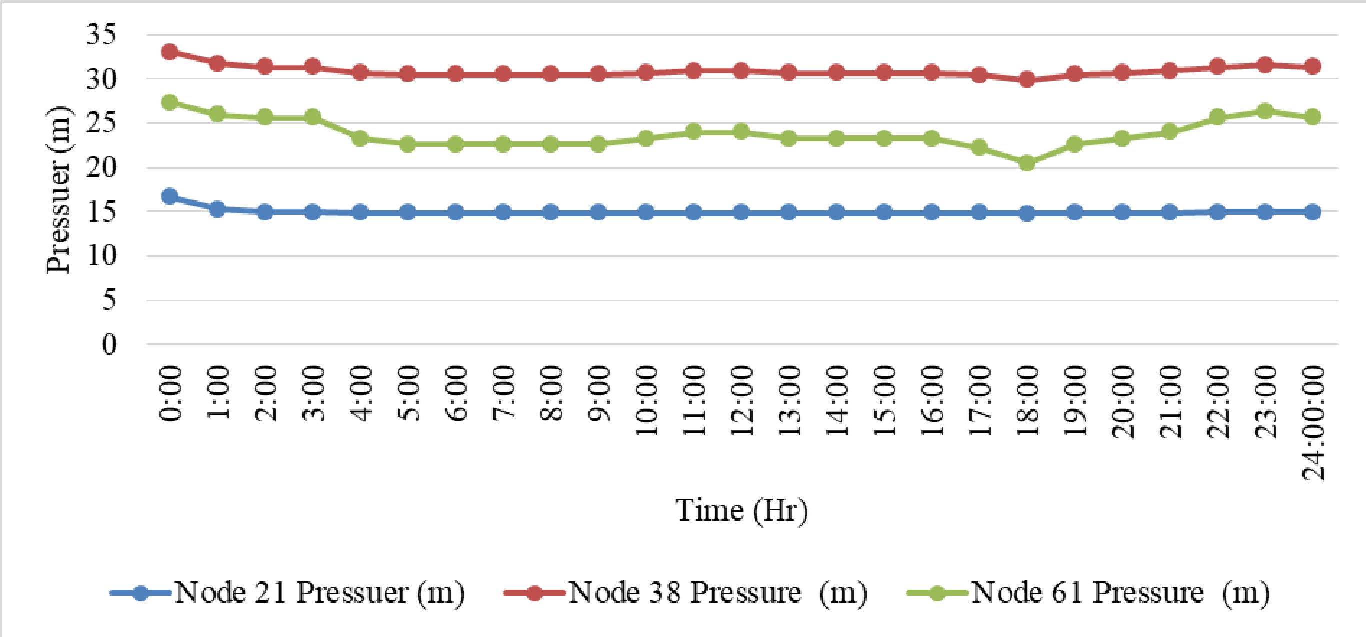

Figure 18 Pressure distribution of selected nodes in zone C of the proposed system

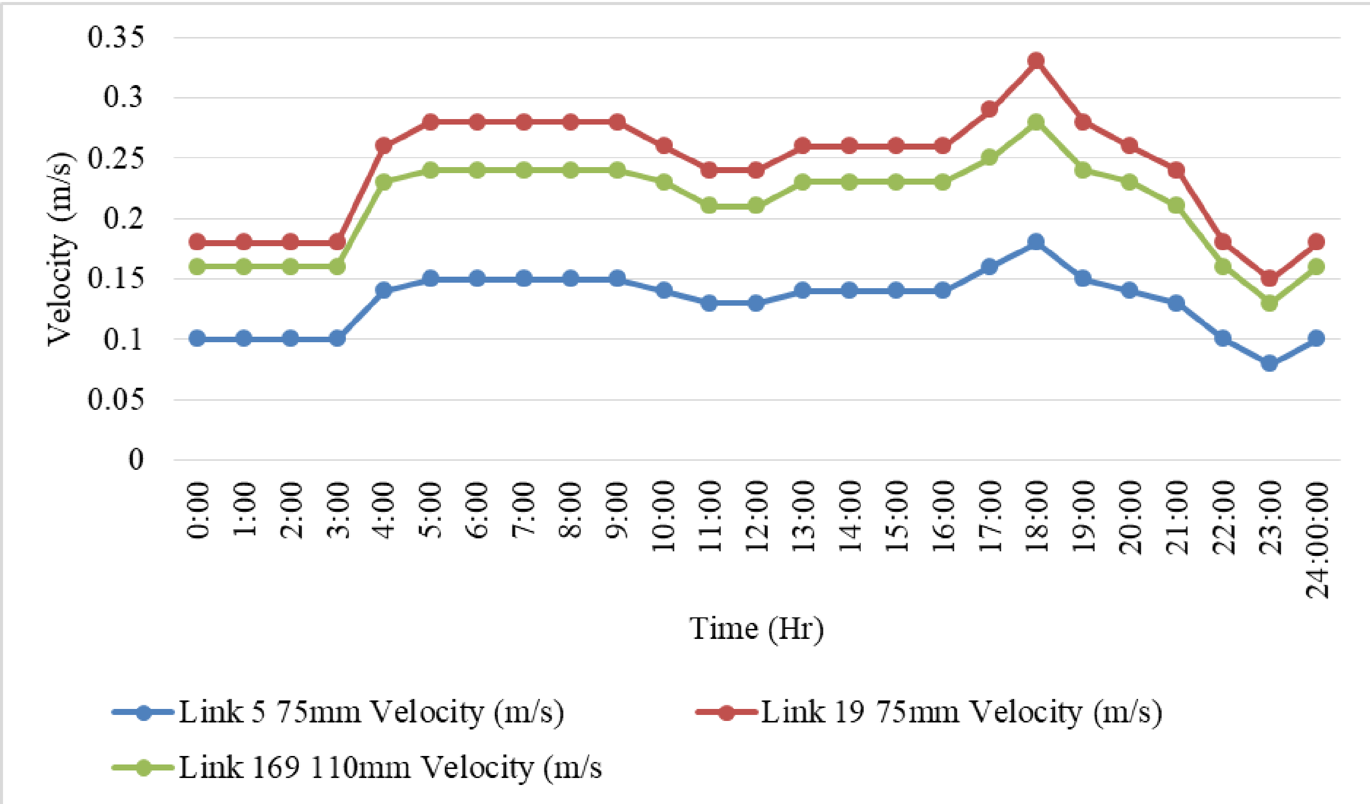

Figure 19 Velocity distribution of selected links in zone C of the proposed system

Head loss distribution of selected links in zone C of the proposed network

Since head loss (H_L) is directly proportional to velocity (V) and discharge (Q) as reflected in Darcy-Weisbach and Hazen Williams equation of headloss, it means that and increase in one variable (velocity) leads to an increased the other variable (headloss). It was obvious noticing that headloss in the selected links in figure 20 above showing the same trend as velocities in figure 19 above. Both velocity and headloss values were constant between 00:00am-03:00am in Figures 19 and 20. The head loss values saw a slight increase as velocity increases in the simulation period of the 24hrs. Head losses, however, were within design standard of 5m/km.

General hydraulic parameters performance of the proposed network

Based on the hydraulic parameters observed in the various nodes and links of the proposed distribution system, the system has the potential of providing high level of service to consumers. It was also observed that the water system was well balanced, and able to provide minimum pressures at all demand nodes, thereby solved the problem of high and low pressure, which could damage pipes and appurtenances or trigger inadequate flow. The analysis of the results to determine areas with similar hydraulic parameters characteristics in the various zones shows the following pressure characteristics, zone A (49.11m -58.01m), zone B (39.37m-47.75m) and zone C (14.76m-33.04m). These pressure ranges can guide operators on flow regulation in the network to satisfied consumers concerns. The peaking hour of 18:00pm across the entire

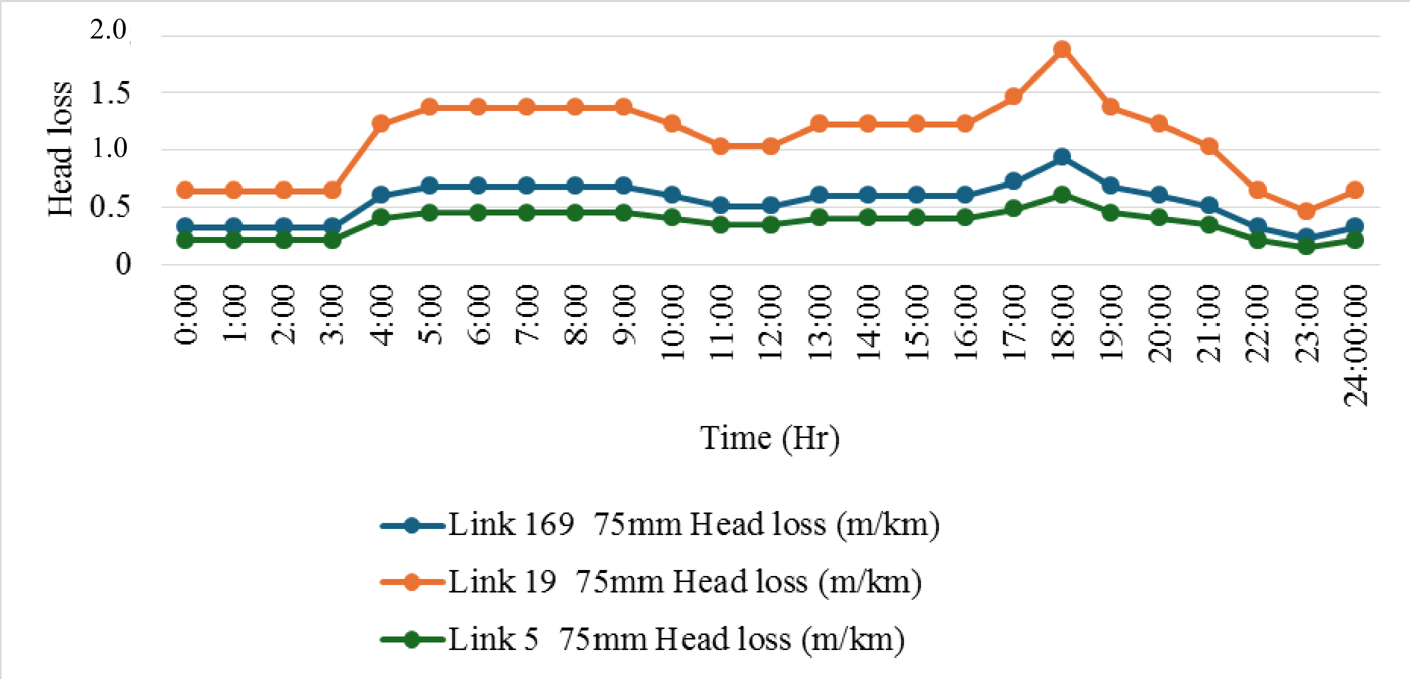

Figure 20 Head loss distribution of selected links in zone C of the proposed system

network also means that majority of the population draw water at 18:00pm leading to a drop in pressure in the network. Head losses over the 24hr simulation period were within design standard in the entire proposed network likewise velocities. This explained the robust nature of the proposed network to withstand or overcome challenges cause by arbitrary introduction of new nodes to satisfy consumers.

Current water coverage and network pipes composition in existing WDS

The current network has a water distribution network coverage of 24.006km with 141 nodes and a total length of 3.5km of transmission network. Seventy percent (70%) of the population had direct access to the network and received their water supply from the water system representing 8,750 inhabitants. The remaining 30 % accessed water from hand pumps and other sources. Some inhabitants walk beyond the recommended distance of 500m to access potable water. A total number of 1,360 households were legally recognized as registered customers from the water system billing software with and average households of six (6.) Out of the 1,360 households, only two (2) were categorized as commercial subscribers because they were using the water to operate washing bays. Ten (10) institutional subscribers and a greater chunk of 1,348 were domestic subscribers. The network structure comprises of polyvinyl chloride pipes (PVC) for both distribution and transmission except for riser pipes for submersible pumps which were high density polyethylene (HDPE) pipes and galvanized iron (GI) pipes laid into structures at the pump houses and at the elevated concrete tank. These pipes were of varying sizes ranging from 50mm to 150 mm.

The network configuration of the distribution system has one loop layout, and the remaining structure is the branched type of network. The primary feeder was 150 mm, and the central part of the town was supplied by 110-mm pipes. However, 90-mm diameter pipes served other portions of the town, whiles 50-mm pipes were mostly linked to standpipes and they represented the smallest pipe diameter in the distribution network. Aside sediment accumulation at dead ends which causes odour and taste problems, there is also difficulties in serving other portions of the town during some distribution pipes maintenance works, thereby depriving customers access to water because of the limited interconnectivity of the network due to its branched nature.

Conclusion

Understanding water distribution network hydraulic performance is primarily important for a water supply system management and operation. The study assessed the existing WDN hydraulics parameters performance using hydraulic based model (EPAnet 2.0) and found that during the twenty-four (24) hours simulation period in the three (3) zones, the existing system run for a maximum of five hours (00:00-05:00am) and negative pressures were the results of the rest of the simulation period which suggested the network inability to deliver the expected demand. Higher head losses and greater variations in pipes velocities were some of the consequences of the inefficiencies observed in the existing network. There were significant head losses of 161.66 m/km in some links, almost thirty-two (32) times higher compared to the small-town sector guidelines recommended standard of 5 m/km. The range of velocity values across the three (3) pressures zones were from 0m/s – 2.81m/s. All the hydraulic parameter (pressure, velocity, head loss and flow) analysed in the existing network did not meet minimum design benchmark because there were very low, and very higher values observe among the hydraulic parameters.

A proposed network was then modelled accounting for the challenges observed in the existing WDN. The remodelled network was balanced with hydraulic parameters (pressure, velocity and head loss) meeting designed standards and peak hourly demand occurred at 18:00 hours across all the three (3) pressure zones. The study recommends further studies on transmission network performance evaluation and rehabilitation of transmission network considering pumps modelling and wells redevelopment. Further studies on the performance of water quality integrity of the water distribution system since the hydraulic performance revealed higher head losses which are signs of water losses, and this has the tendency of compromising water quality. Duty bearers should include deliberate policies towards re-planning study of WDSs to operate with new technologies such as system automation.

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Conflict of Interest Declarations

The author declares no conflict of interest.

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