VOL. 14, NO. 12, JUNE 2019 ISSN 1819-6608
ARPN Journal of Engineering and Applied Sciences
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TYPICAL WATER SUPPLY SYSTEM AND DEMAND BALANCE DESIGN IMPROVEMENT IN A DEVELOPING COMMUNITY

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ABSTRACT

A typical demand balance based improvement design of Water Supply System in a developing Community is presented. Taking data from a characteristic Campus Community in Nsukka, South East Nigeria, hydraulic calculation of water networks was implemented using selected systems of equations that were best suited for the exercise. The EPANET 2 software was explored in performing required pressure, velocity and flow analysis peculiar to the studied Campus. The Water demand of 3, 645,000 liters per day at the time of Campus inception in 1960 had risen to a demand of nearly six million liters per day at present, due to expansion in all spheres of the Campus. The new design was based on present and future Water demands in the Area which were estimated as 5,901,834 liters/day and 55,764,618 liters/day respectively. The result shows that Water supply for all Campus needs can be improved by 60% under the assumption of per capita liter demand of 150 in the improved design.

Keywords: water supply system, demand balance analysis, university community, EPANET software.

1. INTRODUCTION

Food of which Water is an indispensable part is the most important need of all living things. Survival of all mortals is very difficult without Water, and very discomforting when it is supplied in insufficient quality and quantity. The Industry is not spared from sufficient water demands since it is required for most processing and sanitary activities. There is no end to emphasizing the necessity of Water as the central column on which many beams of animate and inanimate systems gains pillars of support. A water supply system (WSS) is a network that consists of different components (pumps, reservoirs, pipes, valves, etc.) that are used to transport drinking water from one or more resource nodes to multiple demand nodes (domestic, commercial, and industrial consumers (Annelies and Sorensen, 2013). It is regrettable that this very important necessity is supplied intermittently in some developing countries, let alone, Campuses in the Countries, if available. There are instances where the inhabitants resort to run-off water that appear to be good upon sedimentation with time. This practice comes with various risks to human life. Water supply must be continuous, and meet both quality and quantity status in all spheres to improve quality of life as well as meet industrial production targets. To achieve this, a water management scheme capable of ensuring all time water availability is inevitable. Sanker *et al* (2015) discuss the disadvantages of insufficient water supply and proposed a Water distribution system, using data from the Kathgarh (Indora) area of Himachal Pradesh. Marques *et al* (2014) listed the danger posed to humanity as a result of increasing demand Water, following alarming growth in population, without a matching increase in Water supply systems. The major objective of the water distribution system is to supply water at a sufficient pressure level and quantity to all its users. Water supply systems can deliver water from reservoirs, tanks and water treatment plants to consumers via an interconnectivity of pipes, valves, bends, taps and rotor-dynamic machines, of which pumps are typical example. The uses and expectations of water distribution networks in homes and industries alike are discussed by past researchers (Herrera-Leon *et al.*, 2019, Sankar *et al*, 2015, Udoibuot, 2016, Corte and Sorensen, 2013). Multi-objective optimization using energy based approaches for sustainable water management have been, implemented (Luna *et al*., (2019, Lima *et al,* 2018). Earlier presentations on multi-objective methodology found application in checking water supply interruptions (Ilaya-Ayza *et al,* 2017). Water supply networks serve many purposes in addition to the provision of water for human consumption, which often accounts for less than 2% of the total volume supplied. The water harvested from pipes can be used for washing, sanitation, irrigation and fire-fighting. Every institution of higher learning must possess a sustainable WSS for maximum productivity. With this great importance attached to Water supply networks, a constant design or redesign of existing systems will remain a reasonable practice. Performance of hydraulic analysis for water networks can be of significant benefit, as it has been shown to impact on water quality (Nono *et al,* 2018). Taking data from Phakalane water system in Botswana, the authors use two quantities, namely; water volume and nodal percentages to investigate the viability of introducing alternate interventions in the operation process of the network, and check its effect on water quality. Improved water quality was said to be possible if the results were implemented, short of using the subsisting intervention at the time of the study.

Many developing Universities in developing economies often find it difficult to access Water when needed. Sometimes, the inhabitants will have to walk a great distance before getting water. This problem is not peculiar to rural dwellers as some municipality in Urban areas are made to face similar problem, especially when

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Urbanization results in uncontrolled Water demand and supply deficit. The existing WSS which was designed for a lesser population can, if not always remain unchanged, whereas the number of end users have more than doubled. The most feasible management strategy in such situation is to model or redesign the Water distribution system in line with prevalent population and industrial realities of such places. Water supply issues in the mould have already been given a recent attention. For instance, Tianhong, *et al* (2019) use a fifteen (15) year water flow data in Shenzhen to simulate all significant interactions that play out during the water cycle. The WSS indices employed in the study include growth in secondary-tertiary industry, surface and wastewater utilization rate, domestic water demand per capita as well as value of added industrial output consumed per unit. The investigation results portends further decrement in usable water availability for the study stretch, following further scenario analysis implemented with the parameters identified above. Other parameters than the ones identified in the presentation immediately above can emanate while improving existing water supply system in other spheres, and necessary simulations can also differ. Researchers have suggested dwelling on the major variables and maybe returning to the ones left out to reduce complications. Zhe *et al* (2015) presents two modelling approaches to WSS. Firstly, macroscopic modelling, in which case, the emphasis is on major variables among water network. Reservoir depths pump flows and pressures at selected pressure points were considered within the macroscopic level. The second approach is employed at the initial design stage where microscopic quantities like the network topology, exact parameters of pipes and nodes as well as a properly estimated nodal demand is the concern. Hydraulic status is an important parameter that can be considered in simulation and improvement effort in WSS. Losses at various stages of the WSS can lead to failure risks, when poorly handled or completely ignored (Pietrucha-Urbanik and Studzinski, 2019). Such losses can affect the performance of a proficiently designed WSS (Nono *et al*., 2018, Gomes, 2011). Certain measures can mitigate Water losses in pipe run, from simple to moderate WSS, which include pressure management, total quality maintenance that emphasize leakage control among other activities. Some scenarios that can aid water supply improvement on the basis of hydraulic simulation had been reported (Afrasiabikia *et al*, 2017). Gomes (2015) suggested full implementation of District Metered Areas (DMAs) aspart of losses control measure in areas where large water networks are involved.

The case of a Campus Community located in a growing Municipality in South East Nigeria is undertaken in this research. There continues to be unprecedented increase in the number of Water demand centers (dormitories, laboratories, residencies, toilet facilities, churches, clinics, Agricultural farms and Industrial clusters) since the establishment of the University in 1960.The aim of this research is to simulate the water realities of the case study, with a view to finding an improved WSS in tandem with population growth and other environmental changes. Attempt was made to integrate a Hydraulic analysis of the entire water system into the study. The authors are not aware of any presentation in this mould conducted for the case study Campus. The research use EPANET 2 as design tool and include storage facilities for complete characterization of demand of Water in the location and exploration of improvement procedure.

2. DESIGN CONSIDERATIONS

Water is generally transported from one source to a destination by three major identified methods (Udoibuot, 2016), which include gravity system, pumping system or a combination of both. This is not without due regard to distribution using water tankers, cans and other manual means. Selection of a particular method depends on economy, suitability and convenience of the procedure. For example, sufficient height differential is needed in distribution by gravity while uninterrupted supply of power is important for recourse to the use of pumps. The material needed for this research are mainly software that can implement the rigorous analysis involved in the work. One of the most important consideration in the design of a WDS is the water demand variation (Trifunovic, 2006). This variable prevents excess or insufficient water supply. As presented in literatures for the estimation of maximum daily demand (Trifunovic, 2006), 10-30% can be added to the average daily demand. Accordingly, peak factor for the daily water demand (K_2) can be given by: 1.1-1.3. The observable peak periods are recorded in the morning and late afternoon. The hourly peak factor (K_2) which lies between 1.5 and 2.5 can be multiplied by the average hourly demand to determine the peak hour demand. The variability of the peak hour demand depends on population and character of the area under consideration. The typical range of velocities in distribution pipes ranges between 0.5 and 1.0 m/s, but can go up to 2 m/s. Other parameters of interest in designing a WDS include population, pressure and density.

2.1 Design Stages

The research followed the modelling steps that are distinguished in literature for WDS (Lima *et al*., 2018). The major design stages include input data collection, network schematic, model building, model testing and problem analysis. Information on network layout architecture, topography, system type and population are normally obtained during data collection. Other variables that can be considered are water demand in the specified location, system operation and maintenance strategies. Hydraulic calculation of water networks consumes a lot of time, especially in selection of systems of equations whose complexity varies almost directly proportional to the size of WDS (Emmanouil and Langousi, 2018). Thus, some Schematic approaches can be necessary up to the level where the model accuracy will not be substantially affected, to permit precise calculations. Several model building strategies have been proposed in recent WDS design (Satori *et al,* 2017). The one which emphasizes progressive increment in the level of detail seems most

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applicable, as beginning with full-size network can result in difficult combinatorial problems during model testing.

2.2 Model Validation

When the model building is completed, it is expected that the models be validated by testing, to be sure that the results matches real life situations. A lot of trials are implemented until logical responses to any input alterations becomes evident in the sense that the physical features of the model has been represented appreciably by the model.

2.3 EPANET Algorithm

The EPANET software employed in this research performs extended period simulation of water quality and hydraulic behaviour within pressurized water distribution pipe networks. It is a Windows based program. The functionality is able to track water flow in each pipe, nodal pressures, tank water head, and chemical concentration along the pipe run during a multiple step simulation period. The power of the software enables simulation of other important features, like the water age and source tracing. The hydraulic functionality of the EPANET can analyze a great size of Water networks. It computes friction losses with the aid of classical models of Hazen-Williams; Darcy Weisbachand Chezy-Manning flow models (Larock et al, 2000). Minor head losses in bends, pipe fittings, pumping energy with cost and speed pumps can all be delineated by the software. EPANET can model storage tanks with varying shapes as well as multiple demand categories at various patterns of nodes. Pressure-dependent flows from sprinkler heads or emitters and complex rule-based controls can both be achieved with the aid of the software.

As presented in literature for the implementation of the EPANET algorithm under the assumption of a pipe network of N junction nodes and NF fixed grade nodes, the flow-head loss relation in a pipe between nodes i and j can be represented as (Rossman, 2000):

$$H_i - H_j = h_{ij} = rQ_{ij}^n + mQ_{ij}^2 \quad (1)$$

Where:

H = nodal head, h = head loss, r = resistance coefficient, Q = flow rate, n = flow exponent, and m= minor loss coefficient, i and j are two different points on the pipe.

The adopted friction head loss model determines the value of resistance coefficient. In particular, the power law form of equation (2) is appropriate for pumps head loss computation.

$$h_{ij} = -\omega^2[h_0 - r\left(\frac{Q_{ij}}{\omega}\right)^n] \quad (2)$$

Where:

h_0 = the shutoff head for the pump, ω = a relative speed setting,r and n are the pump curve coefficients.

The flow continuity around every node should equally be satisfied in the WDS design, according to the dependence of model (3):

$$\sum_j Q_{ij} - D_i = 0 \text{for i = 1, …N.} \quad (3)$$

Where:

D_i = flow demand at node i

It is convenient to assign positive values to flows into a node and negative values to contrary flows. In the design, solution is sought for heads and flows which satisfy models (1) through (3) at the initial stage. Subsequent nodal heads can be obtained by solving the matrix represented in equation (4):

$$AH = F \quad (4)$$

The diagonal elements and the non- zero off-diagonal terms can be represented by equation (5) and equation (6) respectively.

$$A_{ii} = \sum_j p_{ij} \quad (5)$$

$$A_{ii} = -p_{ij} \quad (6)$$

Where:

A = an (N x N) Jacobian matrix
H = an (Nx1) vector of unknown nodal heads
F = an (Nx1) vector of right hand side terms
p_{ij} is the inverse derivative of the head loss in the link between nodes i and j with respect to flow

The inverse derivatives of pipes and pumps head losses can be represented by equation (7) and equation (8) respectively.

$$p_{ij} = \frac{1}{nr|Q_{ij}|^{n-1} + 2m|Q_{ij}|} \quad (7)$$

$$p_{ij} = \frac{1}{n\omega^2 r\left(\frac{Q_{ij}}{\omega}\right)^{n-1}} \quad (8)$$

Equation (9) accounts for th+e flow nodal imbalances at the right hand side (RHS) terms of equations (7) and (8) while the flow correction factors for pipes and pumps are presented in equation (10) and (11) respectively.

$$F_i = \left[\sum_j Q_{ij} - D_i\right] + \sum_j y_{ij} + \sum_f p_{if} H_f \quad (9)$$

$$y_{ij} = p_{ij}\left(r|Q_{ij}|^n + m|Q_{ij}|^2\right) sgn(Q_{ij}) \quad (10)$$

$$y_{ij} = -p_{ij}\omega^2\left(h_0 - r\left(\frac{Q_{ij}}{\omega}\right)^n\right) \quad (11)$$

Where:

yij = the flow correction factor, $sgn(x)$ is 1 if $x > 0$ and -1 otherwise

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Note that Qij is always positive for pumps.

Succeeding flows can be computed by the use of equation (12), which is the flow update rule. Hence, the absolute flow changes relative to the total flow must not be greater than some tolerance (0.001). Otherwise, equations (4) and (12) must be solved iteratively until flow continuity is achieved in all links.

$$Q_{ij} = Q_{ij} - \left(y_{ij} - p_{ij}(H_i - H_j)\right) \quad (12)$$

Full implementation of the EPANET modeling algorithm demands that the intended network representation of the location is available, otherwise the network must first be drawn. The constituting properties of the system can be edited while giving a detailed description of the system functionality. There are certain options that are open to the analyst where a definite analysis system can be selected before running the hydraulic analysis. The last step is an iterative process that can continue until a satisfactory result is obtained. Figure-1 presents the procedural steps followed in implementing the WDS problem.

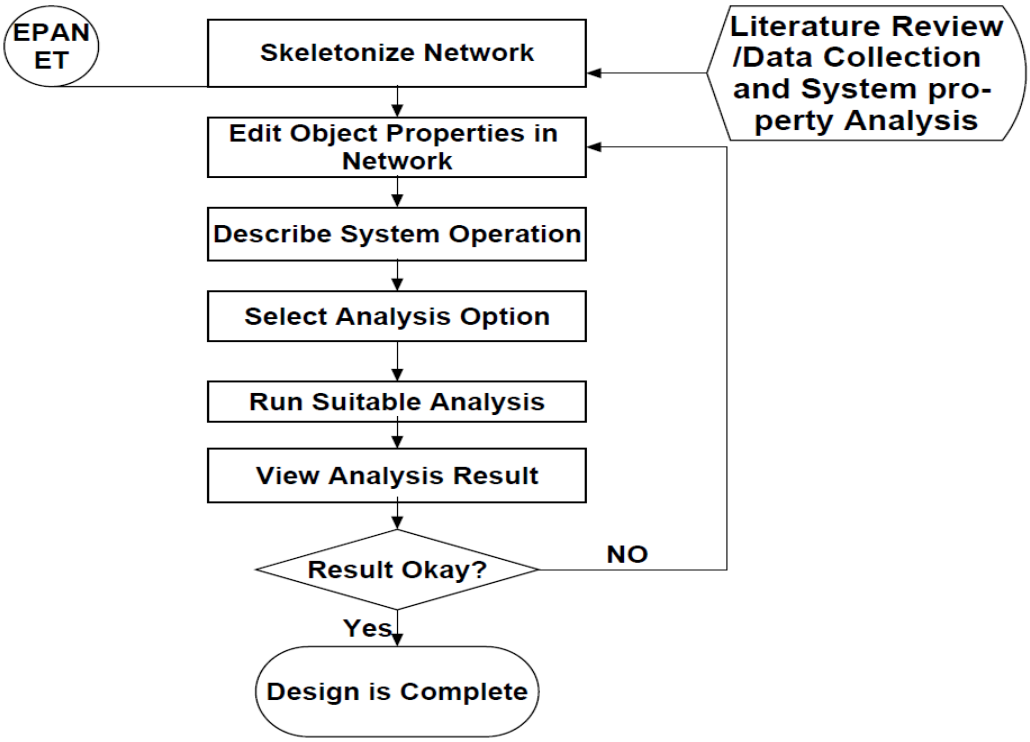

Figure-1. Procedural steps followed in implementing the work.

2.4 Network Skeletonization

There can arise situations where the entire water network becomes so complicated that the detail analysis of every single pipe becomes time consuming as well as reduces clarity. Under such conditions, the representative model can address the complication by utilizing selected pipes and nodes which has significant influence in the network and living the extraneous ones behind. Water distribution systems treated in this way can be said to have been skeletonized (Jung, Boulos, and Wood, 2007). However, the truncated flow or water demand created by this procedure can be amortized by sharing the demand to the nearby pipes along the run of the neglected pipes. This circumstance can be encountered in design of WDS involving supply to houses both existing and impending, especially in villages without a particular housing layout. The major advantage is that it can cater for population expansions and unforeseen structures. The flow in such eventuality can be distributed to some nodes in the intersections found in nearby streets as a result of the skeletonization process. Closely related to this design cautionary exercise is the application of time windows. This accounts for the variability of water demand vis-à-vis various periods of the day. The demand when students are preparing for school and when majority of them are in class are obviously going to differ. This is the same with when most of the students have returned to the residencies in the evening. So a time window that incorporates nodal demand variation at different periods appears very appropriate, to make the designed network both efficient and flexible.

2.5.1 Assumptions

Working with in-situ conditions found in the case study area will generate implementation problem in the EPANET software. In particular, the maintenance culture of the water management is not factored into the design to ensure steady functionality of the pipes and other

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distribution facilities. Hence, this research assume that the distribution facilities are as good as new at all times, such that there are no leakages, to enable a corollary assumption of continuous flow. If there is need for maintenance, whatever practice should be able to restore the system to as good as new or the so called perfect maintenance (Ozor and Onyegegbu 2011, Wang and Pham, 2006). There is also no guide to the extent of water an individual can use in a day available. During data collection, it was observed that students can and frequently does exceed hostel design capacity, through smuggling squatters and relatives at certain periods. This paper assumes that each individual in the Campus can use 150 liters of water per day. It is good to know that there are many students that pay for residence accommodation but decide to live outside the Campus. Many categories of staff do not stay in the campus beyond normal working hours, Therefor, the second assumption makes it possible to have water always, from zero to excess of people in the campus. The population estimation represents an approximate calculation or a parametric value deduced from considered statistical sample and assigned to a population (Wang and Wang, 2017). Average annual growth rate of 3.76% was assumed in line with statistical analysis of the campus growth trend combined with recent growth rate model found in past work (Ahmadalipour, *et al.*, 2019). Pressure head of 17m was assumed after determination of the pressure required to take water over a head of the tallest storey building found in the campus. Literature (Zeghadnia *et al.*, 2019) present appropriate models for computation of friction losses in pipe flow. In particular, particularly Darcy Weisbach and Hazen Williams models among others. While the former thrives well for any liquid or gas, the latter is considered more accurate for pipe sizes 3 inches (75 mm) and above, for water flowing within the temperature range of between 40° and 75ºF (21° to 24°C) and pressures up to 175 psi (1.2 MPa). This explanation informed the choice of Hazen William's model for determination of friction losses in the WSS. The pressure drop which can be represented as in equation (13):

$$Pd = \frac{4.52 \times Q^{1.85}}{C^{1.85} \times d^{4.87}} \quad (13)$$

$$T_d = 0.002082 \frac{100^{1.85}}{C} \times \frac{Q^{1.85}}{d^{1.8655}} \quad (14)$$

$$V = 1.318 \times C \times R^{0.63} \times S^{0.54} \quad (15)$$

$$Q = 0.849 \times C \times A \times R^{0.63} \times S^{0.54} \quad (16)$$

$$f = \frac{6.05 \times Q^{1.85}}{d^{4.87}} \quad (17)$$

Where:

Q = quantity rate of flow, gpm (Lpm)
C = roughness coefficient, dimensionless
d = inside pipe diameter, in. (mm)
f = friction head loss in ft. hd./100 ft. of pipe (m per 100m)
Pd = pressure drop, psi/100 feet of pipe
R = hydraulic radius, feet (m)
V = Velocity, feet per second
A = cross section area, in (mm)
Td = Total drop in system, psi
L = Total length of pipe run, ft

3. ANALYSIS

Water Supply Systems have been modeled as a set of links connected to different nodes in the past two decades (Rossman, 2000). While the various links can represent components of WSS such as pipes, control valves, emitters and pumps, other parts of the system like junctions, tanks, and reservoirs can be represented by the nodes. Junctions refer to where the various links join. Water may enter or leave the WSS at junctions. The pipes convey water to and from one point in the network to the other. Reservoirs with some accessories function as Water Bank in the system. As nodes, the reservoir represent an extremely large external source or sink of water to the network. Reservoirs are of various sizes, ranging from reinforced concrete structures to large Water bodies like lakes, bays, rivers, aquifers, groundwater and tie-ins to other systems. The pumps raises the hydraulic head of a fluid by imparting mechanical energy to it. When the nodes possess storage capacity such that the quantity (volume) of the Water stored in it maintain a time dependent variability, the node is referred to as a tank. To model the flow through an orifice or nozzle in the WSS effectively, devices called emitters which are normally inserted at junctions can be used. Emitters can equally model flow through irrigation networks and sprinkler systems as well as simulate leakage in a pipe connected to the junction with an estimated pressure exponent and coefficient of discharge for the leaking point. Fire flows (due to low pressure) at the junction can be computed by means of the emitters. The flow (pressure) at determined points of the Water network can be restricted (lowered, increased or stopped) by means of the valves.

3.1 Designing for Peak Water Demand

The water demand is constant throughout the day. Generally, the demand in University of Nigeria, Nsukka is lowest during the night and highest during morning and evening hours of the day (Udoibuot, 2016). The peak hour demand can be expressed as the average hourly demand multiplied by the hourly peak factor. For a particular distribution area this factor depends on the size and character of the area served. Usually, the hourly peak factor is chosen in the 1.5-2.5 range (Trifunovic, 2006).

3.1.1 Sustainable Water Distribution Network

The approaches to the design of Water networks due to Salgado et al (1988) has proved good in many design situations. The solution of flow continuity and headloss along the pipe run can be represented by gradient based equations according to the dependence of equations (19) and (20) respectively.

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$$\dot{Q} = \int_A v dA = V_1 A_1 = V_2 A_2 \quad (18)$$

$$h_f = f \frac{L}{D} \frac{V^2}{2g} = f \frac{L}{D} \frac{Q^2}{2gA^2} \quad (19)$$

Where

$\dot{Q}$ =volumetric discharge through a pipe cross section
V = meanvelocity of the product
A = cross sectional area of the pipe.
f = friction factor
L = length of pipe
D = pipe diameter
Q = discharge
A = Area

Another important continuity principle employed in Water network design is the work-energy principle, which can be written between two sections in a given streamline (Larock *et al.* 2000):

$$\frac{V_1^2}{2g} + \frac{P_1}{\gamma} + z_1 = \frac{V_2^2}{2g} + \frac{P_2}{\gamma} + z_2 + \sum h_{L_{1-2}} - h_m \quad (20)$$

Where

$\frac{V^2}{2g}$ = velocity head
$\frac{P_1}{\gamma}$ = pressure head
z = elevation head
$\sum h_L$ = head term loss
h_m = mechanical energy per unit weight added to the flow by hydraulic machinery

Compliance to the dependence of equation (20) is necessary for steady, one-dimensional flow of a liquid in a pipe in terms of a unit weight of fluid. Equations (18) through (20) can be employed in characterization and deduction of hydraulic status of the Water network at any time, especially with regards to the pipe network.

3.2 Pipe Network Analysis

Three distinct system of equations can be deployed for the solution of pipe network analysis problems, according to which of the flow quantity is the unknown in a particular case. In the first category, the unknown quantity can be the discharge (Q). The H-equations refer to cases where the nodal heads are the unknown quantities whereas the corrective discharge (ΔQ) are sought in the third category. The equations are presented in equations (21) through (24).Following the continuity principle and neglecting losses:

Total flow into a junction = Total flow out of the junction

$$Q_{Ji} - \sum Q_i = 0 \quad (21)$$

Where

$Q_{Ji} = flow\ out\ demand, Q_i = flow\ in\ from\ pipe\ i$

From work energy theory;

$$\sum h_{f_i} = \sum K_i Q_i^n = 0 \quad (22)$$

Where:

$h_{f_i} = head\ loss, K_i, n = coefficients$

$$\left[\frac{H_1 - H_2}{K_{12}}\right]^{\frac{1}{n_{12}}} - \left[\frac{H_2 - H_3}{K_{23}}\right]^{\frac{1}{n_{23}}} = QJ_2 \quad (23)$$

$$Q_i = Q_{01} + \sum \Delta Q_k = 0 \quad (24)$$

3.3 Application to Illustrative Case

The study area is the University of Nigeria, Nsukka campus located in the City of Nsukka in South Eastern Nigeria. The Campus lies between 6°50′47’’N 7°24′09’’E and 6°52’25’’N 7°26’13’’E with an average elevation of 438 m above mean Sea Level. According to the data from Academic Planning Unit, 29, 098 students and 6138 staff inhabited the Community as at 2013/2014 academic session. Figure-2 and Figure-3 presents the study case and the existing water distribution network respectively.

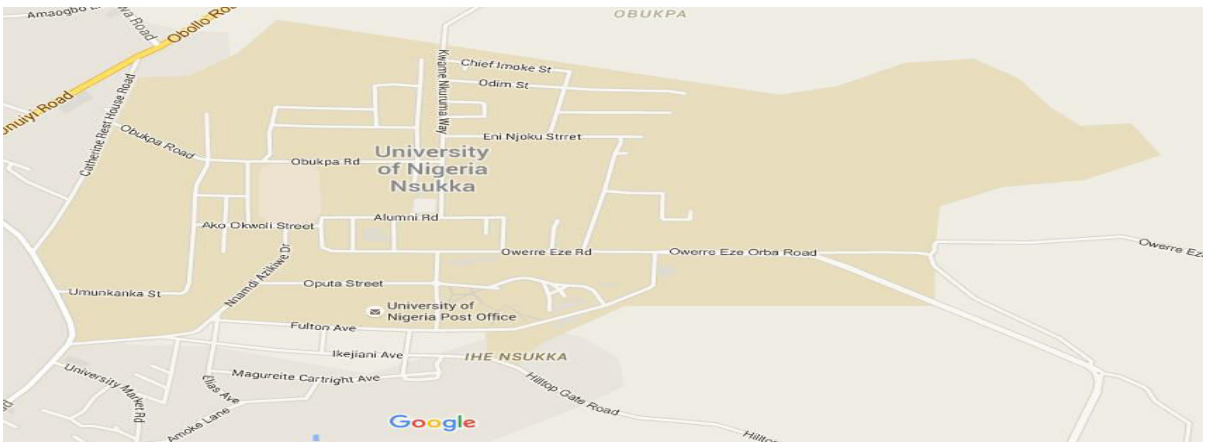

Figure-2. Map of Case Study Campus, Source: Google Map.

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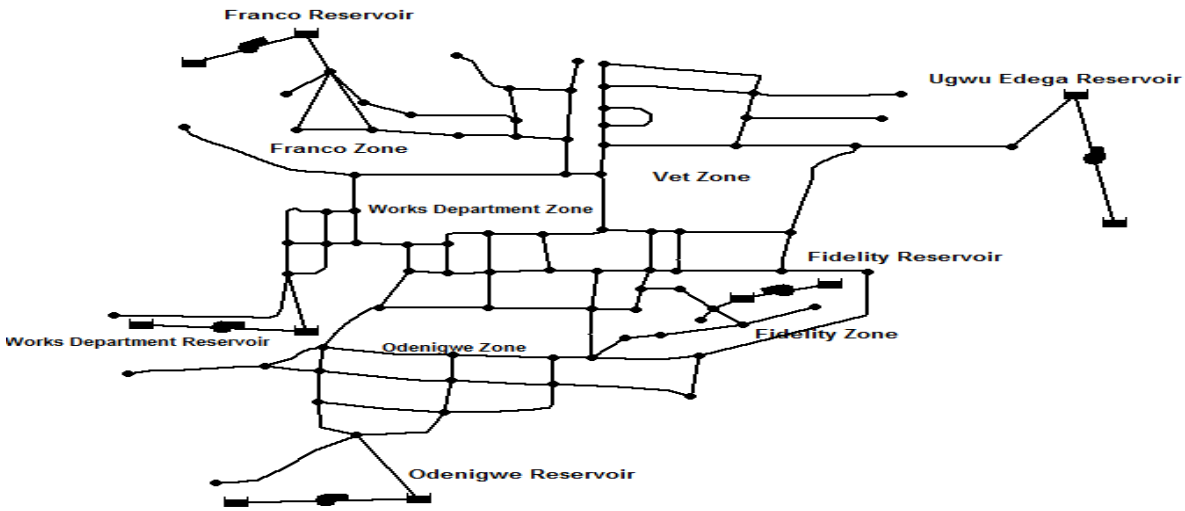

Figure-3. Campus Water supply system.

3.4 Data Collection

The data used for this work include population data obtained from Physical Planning Unit (PPU), University layout, existing WDN and photogrammetric images sourced from Google Earth. Others include student population data sourced from the Academic Planning Unit (APU). The University main source of water is boreholes drilled in strategic positions to enhance water distribution. The reservoirs draws directly from the boreholes and stores the water for distribution. The stored water is treated a-pri-ori to ensure that proper deionization, Chlorination and pH correction is implemented. There is an underground reservoir for collecting clear and treated water before finally transmitting to the reservoirs. There are ten identified reservoirs in the Campus. Two number reservoirs each in Franco, Fidelity, UgwuEdega, Works Department and Odenigwe zones. The capacity of the reservoirs ranges from 688m^3 (Odenigwe hill, Fidelity and Franco) to 909m^3 (Works department). The UgwuEdega reservoir can contain 1360m^3 of Water. The reservoirs are situated between 446m and 510m respectively above sea level. The height differential between the reservoirs and the neighbouring Campus environment makes initial distribution by gravity a good choice. The two reservoir located at Odenigwe and Works department have heights of 6.1m while that of UgwuEdega reservoir has a height of 7.5m. About 10 number boreholes were identified during field visits. Four out of which two are situated around Franco area supplying the works reservoir. The six other bore holes are situated at Safko Construction Company (S.C.C) water generation yard. Only three were being utilized at the time of this research, and were supplying twin Odenigwe reservoir and Edega reservoir. The population data of both staff and student from 1960 to 2013 was obtained from the academic planning unit. The total population of the University serve as the major criteria for the design of a sustainable Water supply system that meets the present and future need of the community. The population was projected for a 50-years design period. In order to project the population for the next 50 years, population growth rate was calculated using model (25) (Oregon, 2002).

$$Pop_{future} = Pop_{present} \times (1 + i)^n \quad (25)$$

Where:
Pop_{future} = future population
$Pop_{present}$ = present population
i = growth rate
n = number of years.

The study of the different activities in the Campus reveals that the main areas of water usage includes but not limited to: *1)* Domestic use - this is classified into residential and non-residential use. The domestic water demand accounts for 50% of total water demand, *2)* Industrial and commercial use - which involves the laboratories and workshop, different commercial and experimental farms, Lodging Centers, Medical Centre as well as the water bottling company simply known as Lion Water. This accounts for approximately 25% of the total water demand as well as *3)* Public use that has to do with fire-fighting, sanitation and construction activities. This account for 20% of the total water demand. In addition, estimation of present and future water demands warranted the use of the guidelines due to World Health Organization (WHO), which stipulates a litre per capita per day demand of 150 as appropriate for university application. Table-1 presents results of current and future Water demands in the Campus.

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Table-1. Estimated Current and Future Water Demands in the Campus.

	Current Water Demand (l/d)	**Projected Water Demand for 50 years (l/d)**
Students	1,187,100	27,634,500
Staff and Households	4,667,250	28,035,150
Lab. & Med. Centre	24,984	49,968
Lodging Centers	17,400	34,800
Swimming pool	5100	10,200
Total	5,901,834	55,764,618

3.4.1 Properties of the Physical Components

The specifications of the components, especially large sized members of the network were ascertained. Specifically, the reservoirs (RSV) and the boreholes (BH). The reservoirs all possess cylindrical geometry. The entire network has equal number (five) of both. A fairly uniform diameter of 250mm was observed for the boreholes. It was interesting to note that the underground water depth was less the actual depth of the boreholes within the neighbourhood of 20m. The reason is to ensure steady supply of water and also to forestall any event of the very expensive pumps used to suck the water from running dry for possible variations in the water levels down the pipe tip. The actual borehole depth of 219.5m was recorded in all the five centers. Other specifications of the reservoirs and boreholes are presented in what follows.

1. Reservoirs

RSV1:	Capacity	12,360,000litres	Minimum Diameter	75m
	Maximum Diameter	97.5m	Design Diameter	95m
	Pressure Head	9.63m	Elevation	478m
	Total Head	487.63m		
RSV2:	Capacity	9,090,000$litres$	Minimum Diameter	62m
	Maximum Diameter	74.5m	Design Diameter	72m
	Pressure Head	8.8m	Elevation	435m
	Total Head	443.8m		
	RSV3:	Capacity	11,376,000$litres$ Minimum Diameter	76.5m
	Maximum Diameter	85.9m	Design Diameter	84m
	Pressure Head	10.9m	Elevation	477m
	Total Head	487.9m		
RSV4:	Capacity	9,360,000$litres$	Minimum Diameter	40.5m
	Maximum Diameter	43.9m	Design Diameter	43m
	Pressure Head	9.0m	Elevation	400m
	Total Head	409m		
RSV5:	Capacity	9,987,000$litres$	Minimum Diameter	45.5m
	Maximum Diameter	57.9m,	Design Diameter	55m
	Pressure Head	9.0m,	Elevation	384m
	Total Head	393m		
	Total Reservoir Capacity	52,173,000$\boldsymbol{litres}$		

2. Boreholes

BH1:	Top elevation	442m
	Bottom Elevation	222.5m
	Total Head	422.5m
BH2:	Top elevation	405m
	Bottom Elevation	185.5m
	Total Head	385.5m
BH3:	Top elevation	453m
	Bottom Elevation	233.5m
	Total Head	385.5m
BH4:	Top elevation	480.5m

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	Bottom Elevation	261m
	Total Head	461m
BH5:	Top elevation	406.5m
	Bottom Elevation	187m
	Total Head	387m

3.5 Hydraulic Analysis of the Entire Network

From the data analysis, the calculated information about the current and future water demands were used to design the water distribution network using EPANET. The results obtained are discussed below. Figures 4 and 5 show the new water distribution network for University of Nigeria, Nsukka Campus with the link IDs and node IDs respectively.

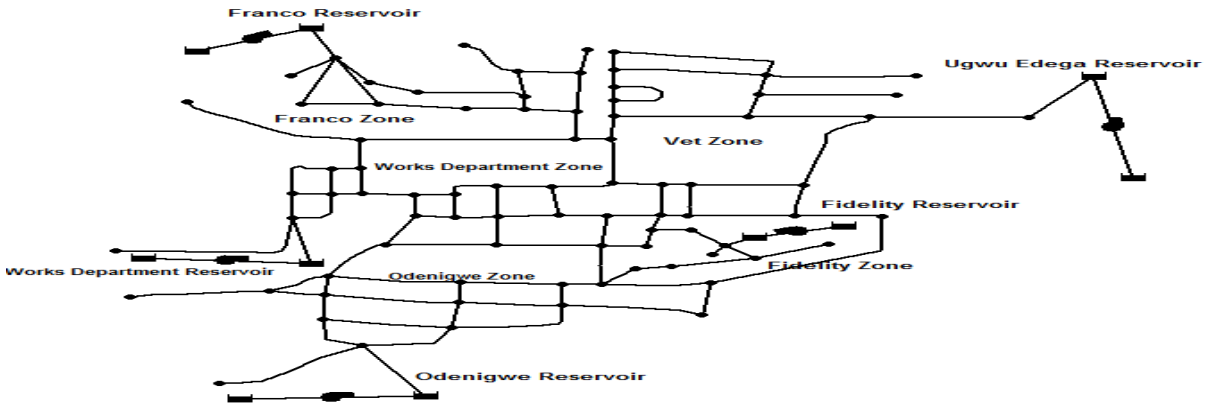

Figure-4. Water Supply System Layout.

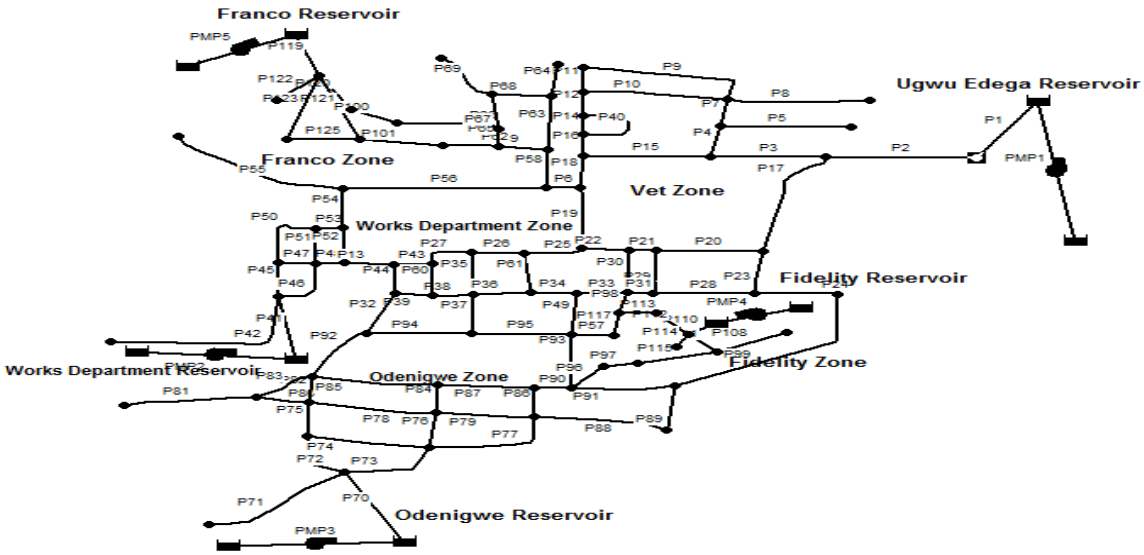

Figure-5. Water supply system with Links.

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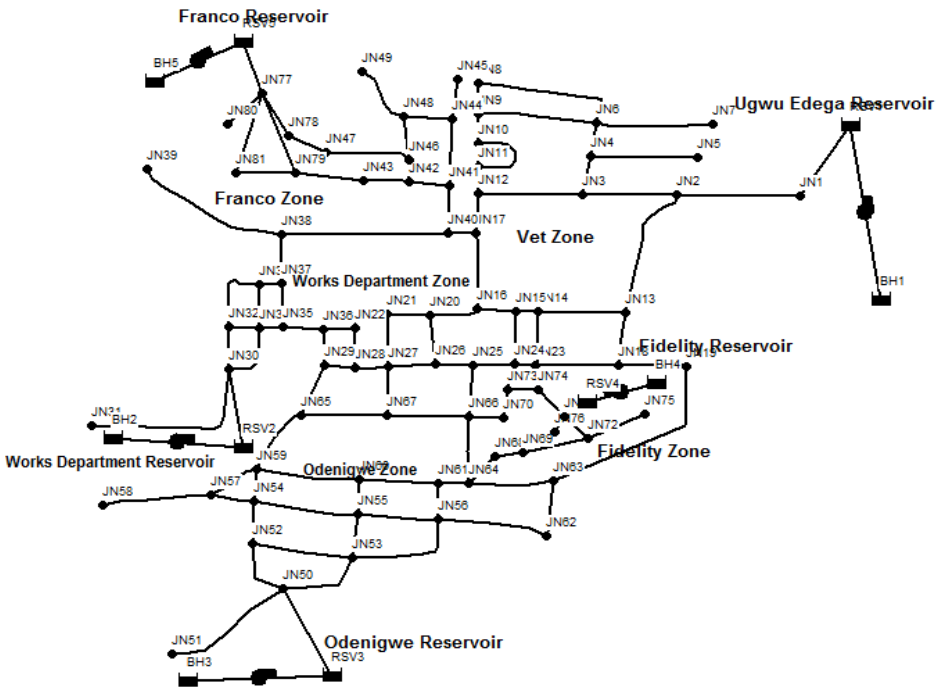

Figure-6. Water supply systems with Nodes.

4. RESULTS AND DISCUSSIONS

The physical structures (Buildings, overhead tanks) in the studied Campus are mainly one or two storey, though upcoming higher structures exist. The EPANET analysis revealed that all the nodes have a pressure head greater than the minimum allowable pressure head of 17m. The results from EPANET for velocities are shown in the figures below. Figure-8 shows that the velocity in all the links in the water distribution network is below 3.0m/s. The results for the flow rates in the water distribution network are shown in the Figure-9. It can be observed that the flow rate in most links is over 0.3lps.

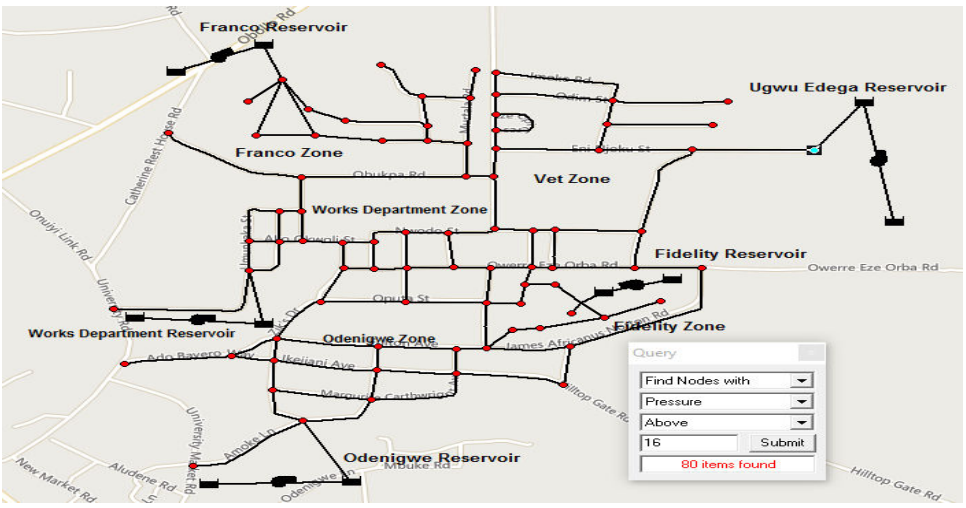

Figure-7. Result showing nodes with pressure above 16m.

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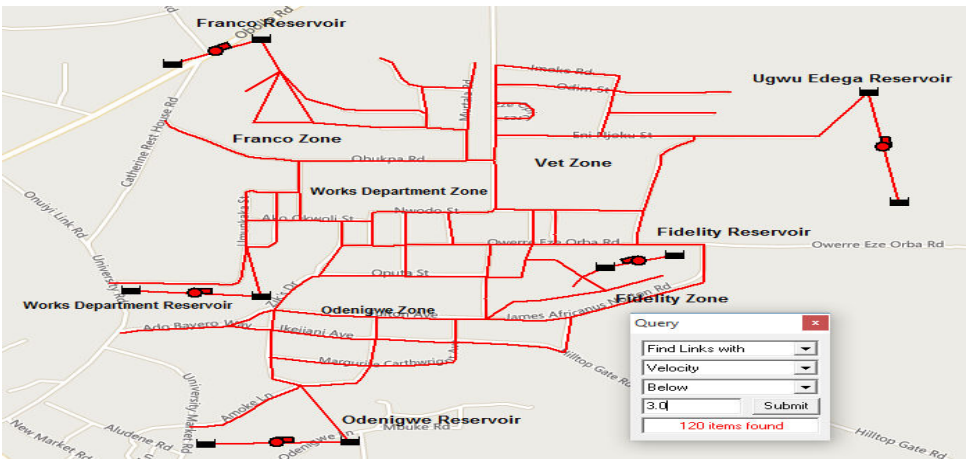

Figure-8. Result showing links with velocity below 3.0m/s.

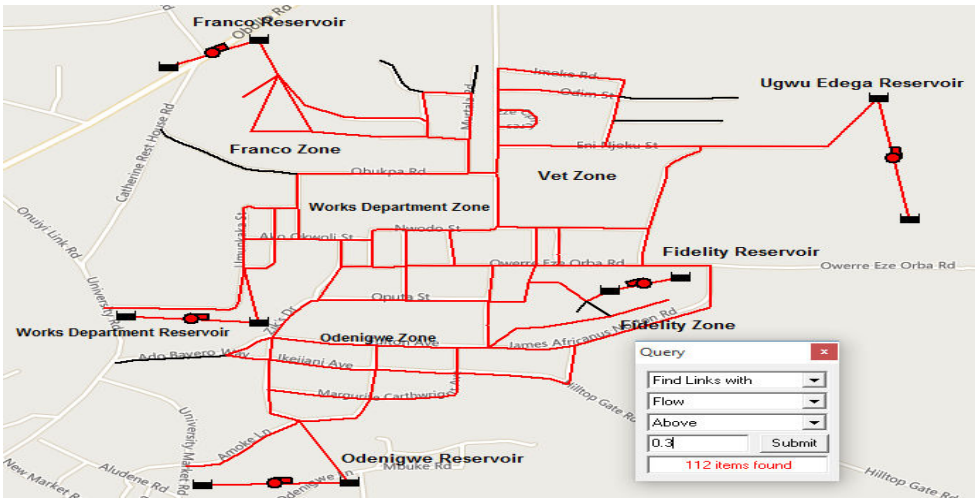

Figure-9. Result showing links with flow rates above 0.3lps.

4.1 Pressure at Nodes across Different Zones for Extended Time Simulation

Some of the infrastructure in the Campus with water retaining capability (stand-alone overhead tanks and containers for Water distribution in buildings) are located in support structures of about 16m. In order to supply water to all the demanding sections of the Campus, a minimum pressure head of 17m was chosen on the basis of interview with Water managers in the Campus. Figure-10 shows the pressure at nodes JN12, JN17, JN16, JN4 and JN9 within Vet. Zone. It indicates that the magnitude of the pressure is not below 17m in order to supply water to areas with multi-storey buildings in the University community. Pressure for selected nodes in Fidelity, Franco and Odenigwe zones as well as Works department area are presented in Figure-11 through Figure-14.

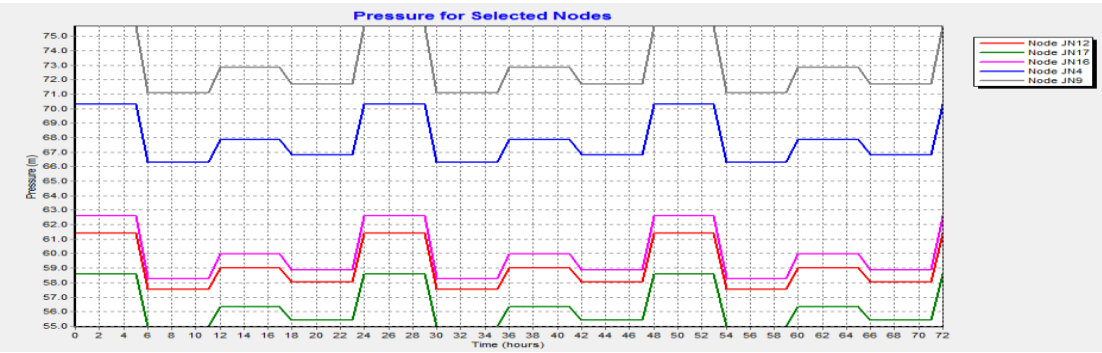

Figure-10. Pressure for selected nodes in Veterinary Zone.

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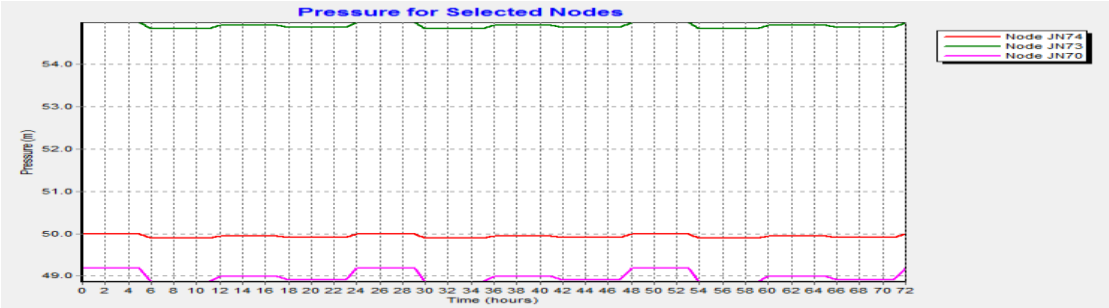

Figure-11. Pressure for selected nodes in Fidelity zone.

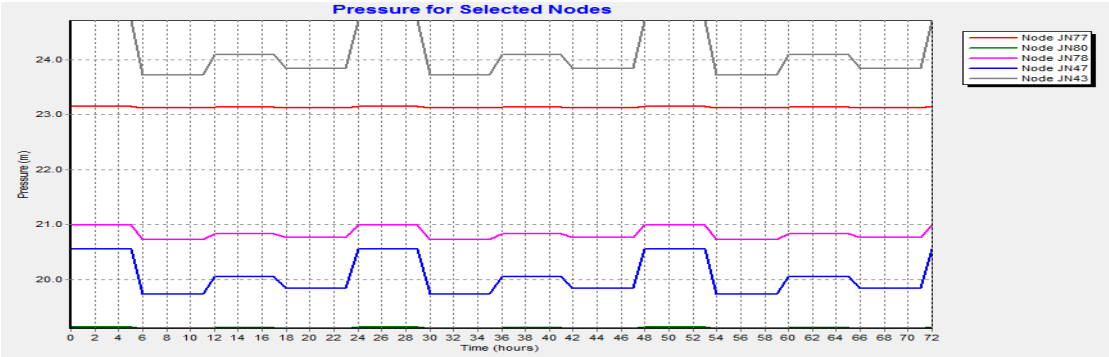

Figure-12. Pressure for selected nodes in Franco zone.

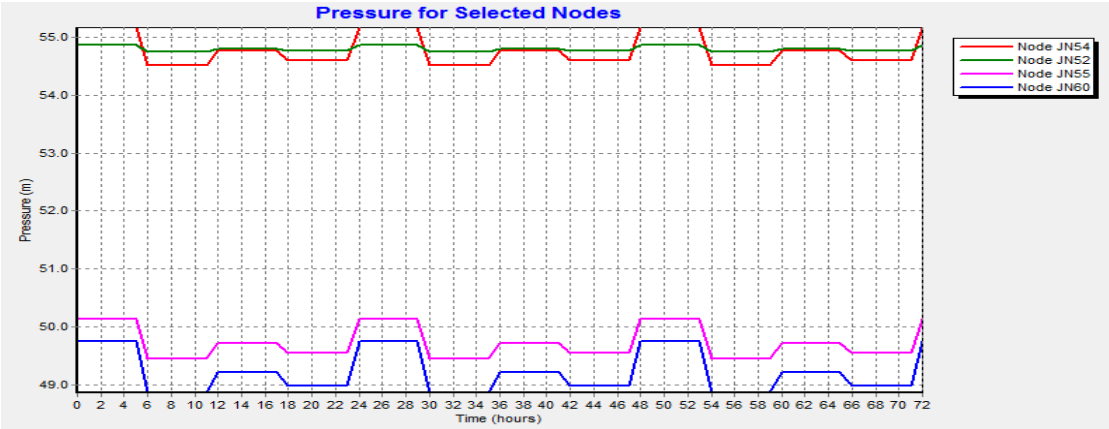

Figure-13. Pressure for selected nodes in Odenigwe zone.

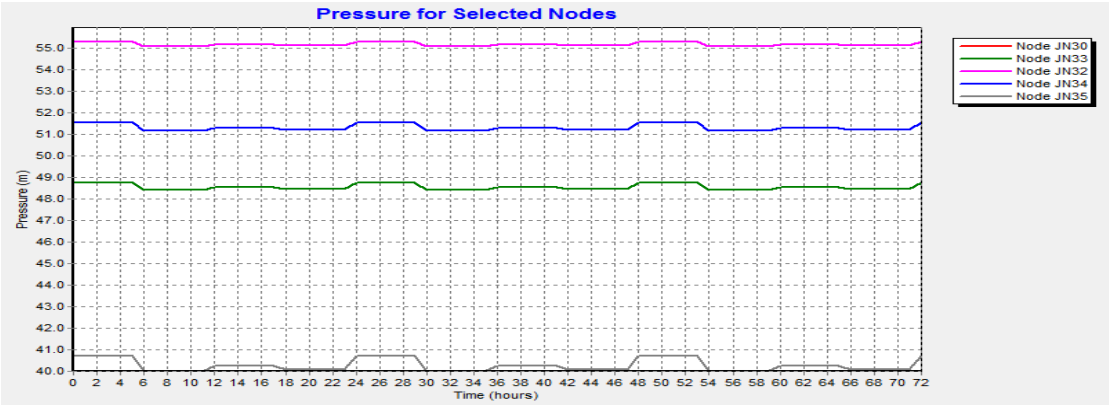

Figure-14. Pressure for selected nodes in Works department zone.

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4.2 Flows in Major Pipelines across Different Zones for Extended Time Simulation

Flow in the links were determined for all the available pipe run, though samples of some of the results are presented. For instance, the flow in pipes P4, P7, P12 extracted from the Vet. Zone are presented in Figure-15. It can be observed that the flow pattern completes a cycle in every 24 hours. Care was exercised to ensure that the magnitude of the flow rate at pipes are not below the recommended 0.3 lps. Flow for selected pipelines in Franco and Odenigwe zones are shown in figure 16 and Figure-17 respectively.

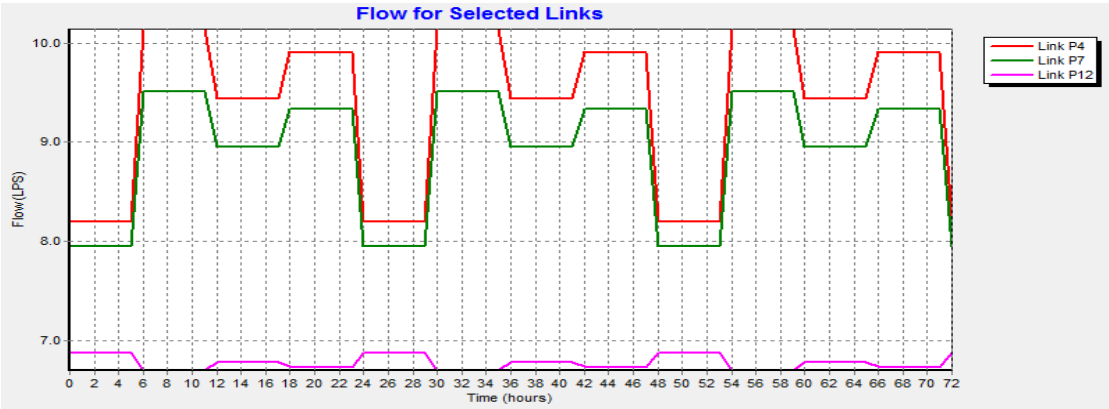

Figure-15. Flow for selected links in Veterinary zone.

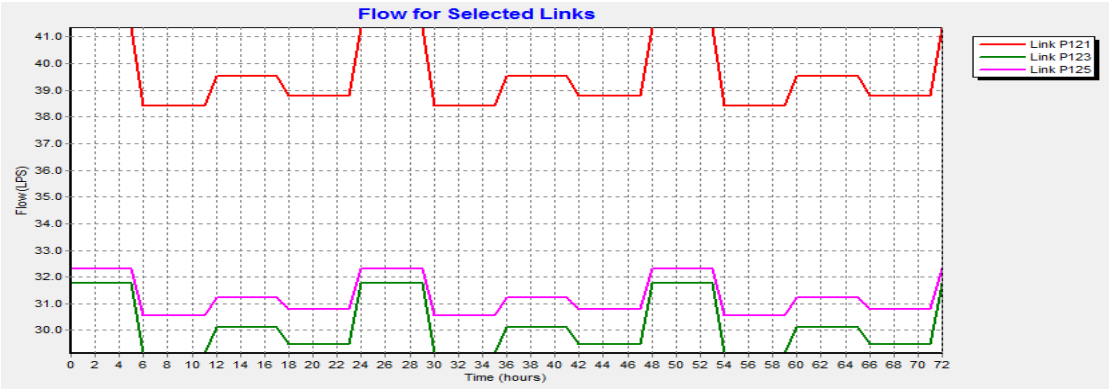

Figure-16. Flows for selected links in Franco zone.

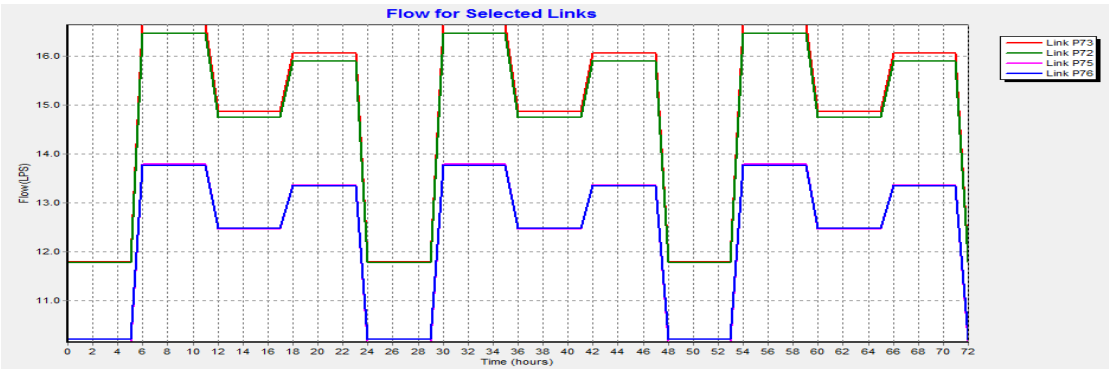

Figure-17. Flows for selected links in Odenigwe zone.

4.3 Velocity in Major Pipelines across Different Zones for Extended Time Simulation

The velocities in most of the pipelines were estimated. The results show that the design recommendation were maintained across every link. For illustrative purposes, the velocity in pipe P45, P46, P50 and P52 within Works department zone is presented in Figure-18. From the graph below, it can be deduced that the maximum recommended velocity of 3.0m/s have not been exceeded. The same applies to graphs for other selected pipelines in different zones in the network. Figure-19 present simulated Water velocities in Fidelity zone.

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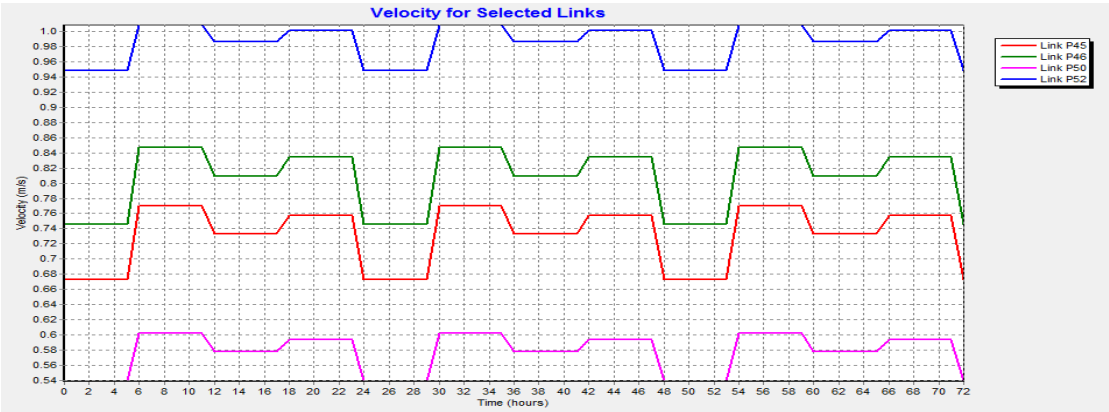

Figure-18. Velocity for Selected links in Works Department Zone.

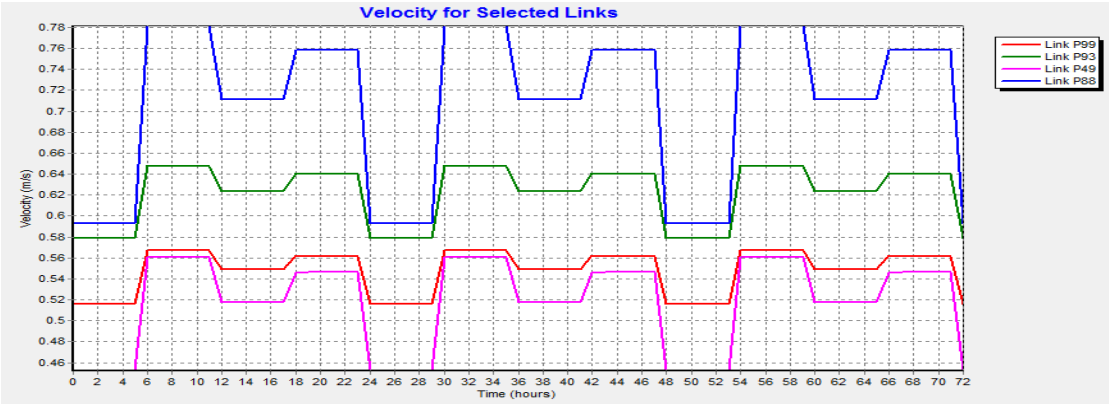

Figure-19. Velocity for selected links in Fidelity zone.

4.4 Demand at the Reservoir

The Water demand due to the redesigned system was also determined and simulated. It is clear that the amount of water produced by the new water distribution system surpasses what obtains in the existing system. This can be pictured in Figure-20. Thus, the network can supply the current demand and future demand in the University community that may be warranted by further expansion in staff and student population, or other areas of Water applicability.

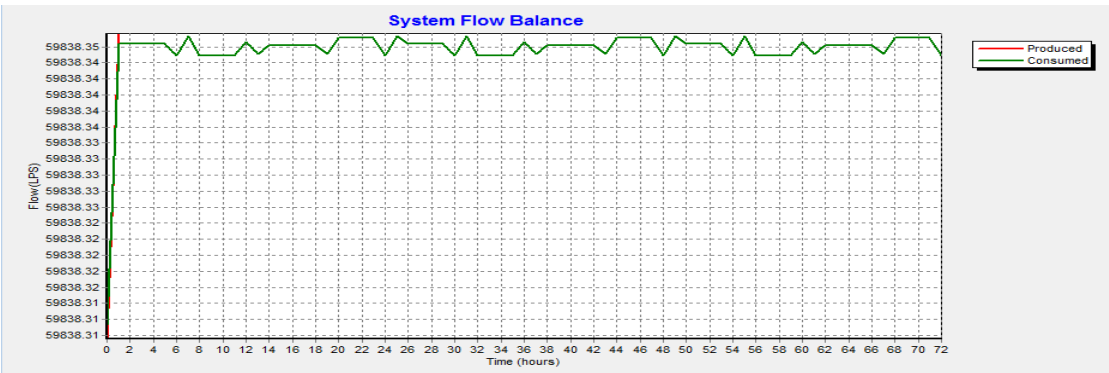

Figure-20. System flow balance.

4.5 Energy Report

The cost implication of the new Water supply system was not left out during the design. The cost was estimated in terms of Energy demand of the pumps that are needed to ensure efficient Water distribution. Table-2 presents the report of energy demand and cost analysis. It shows that the total cost of supplying water to the University community for uninterrupted 24 hours or one day would be approximately twenty thousand, six hundred and thirty USD ($20, 630.00) at a demand of about 52,000,000lpd.

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Table-2. Energy Report of Water Distribution Network.

Pump	Percent utilization	Average efficiency	KW-hr/m^3	Average Power demand (KW)	Peak demand per pump (KW)	Cost/day ~~(₦)~~
PP3	100	75	0.21	2666.67	PP3	
PP4	100	75	136.15	5999.98	PP4	1089280
PP5	100	75	0.2	666.67	PP5	2450871
PP6	100	75	135.97	1333.33	PP6	272320
PP7	100	75	136.26	2666.66	PP7	544638.1
PP8	100	75	136.31	3333.32	PP8	1089276
PP1	100	75	0.24	176	PP1	1361595
PP2	100	75	135.97	1333.33	PP2	71892.48

5. CONCLUSIONS

The major thrust of this work is to present a design and hydraulic analysis of Water supply system in developing public Institution. This feat has successfully been achieved using the capabilities of EPANET Software and constitutive models of fluid flow. The method employed in the work was clearly narrated. An illustrative example of the method was elaborated on data taken from a typical sub-Saharan Campus Community in Nigeria, namely; UNN. In the case example, Water supply facilities installed during the inception of the Campus in 1960 was estimated to be 5, 854, 350litres/day assuming 150 liter per capita Water demand. This estimate had been over stretched beyond its installation capabilities and capacities due to expansion in the staff and students population as at 2015. The results of this research shows that a holding capacity of over 51,000,000liters/day can be achieved at full capacity. The research results can also satisfy the current water demand in the Campus, estimated to be 5,901,834litres/day under the same assumption of 150 litre per capita Water demand. The designed water distribution network will also satisfy the future demand of more than 55,764,618 litres/day.

ACKNOWLEDGEMENT

The material and financial assistance of the NRF-TWAS fellowship: award number PD-TWAS160531166951; UID: 105554, towards this presentation are hereby acknowledged. The management of Department of Mechanical Engineering of UNN and the University of Nigeria, Nsukka are hereby acknowledged for the data and logistic support without which the work would not have been completed. We also thank the publishers of the various online and literature materials from where we sourced necessary data to improve the quality of the work. However, opinions expressed and conclusions arrived at, are those of the authors and are not necessarily to be attributed to the NRF-TWAS, department of Mechanical Engineering of University of Nigeria or UNN in general.

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