ANALYSIS OF BIDA WATER DISTRIBUTION NETWORK NIGER STATE, NIGERIA

Bida Town is undergoing rapid expansion and lack adequate supply of potable water which has given room to the use of different water sources and sometimes results in outbreaks of water borne diseases. EPANET system analysis of Pipe Network was used to analyze the distribution network for both present and future demand after collation of the population data, pipe lengths and diameters, elevation and water demands. EPANET is a computer program based on Hardy cross method of water flow analysis that performs extended period simulation of hydraulic and water quality behavior within pressurized pipe networks. The study has shown that the present situation of Bida distribution network is inadequate in meeting the demand of the consumers, as the outcome of the simulation shows negative pressures in 5 of the nodes while 9 of the pipes has low velocity and 10 pipes recorded high headloss. Areas affected by these inadequacies are those along loops 6 and 7 which include G.R.A, Ramatu Dangana axis and along Bida Town Hall on Ahmadu Bello way. Corrective measures were, however, outlined based on the results. These include modification of diameter of some pipes, provision of thrust blocks and installation of washouts. Future design period of 30 years was also considered and analyzed. Both hydraulic and urban development measures should be used to address inadequacies in the distribution system under future scenario.

CHAPTER ONE

1.0 INTRODUCTION

1.1 Preamble of the study

The provision of water to any community is very important. Among all the abundant natural resources water is the most prominent and essential gift. It is considered most important and inevitable to all earthly creatures due to the properties associated with it. As the world population is growing rapidly water resource of the world is becoming scarce and insufficient, as there is no corresponding increase in the resource.

We need water for domestic activities and plants need the right quantity of water at the right time. We also need water for navigation and hydroelectric power generation. The loss of 20% of body water can cause death. It is possible to survive for various weeks without food, but it is not possible to survive more than a few days without water. Water is life. Access to safe and affordable water is critical to the development of a nation (Jimoh, 2010). The provision of water has a lot of economic benefit to a community i.e. small scale industries like packaged/sachet water, block making industries e.t.c. are encouraged which also attracts population.

Water has special characteristic that is critical to social and economic activity. This has granted it a special status in past and in the modern era, in public policy. Traditionally, freshwater sources have been considered as something in which all members of the human community have rights (Saibu, 1997).

The ability of existing and proposed water supply system to operate satisfactorily under the wide range of possible future demand and hydrological condition is an important characteristic. The likely performance of water is often described by the mean variance of benefit, pollutant concentrations or some operating variables. These performance measures are used in the selection of water supply system capacities, configuration operating policies and target (Cairncross, 1981).

The Federal Government of Nigeria is concerned with the articulation of broad policy guidelines and co-ordination for nationwide water supply and sanitation. The present water supply to urban and rural areas is essentially the responsibility of the various state governments. Sometimes there are Federal Government interventions, especially in rural water supply as reflected by National Borehole Programme and others.

Water resources are limited and highly variable. Pressures from population and development are increasing. Land use and land cover changes have been significant. Current or proposed future water management schemes are large in relation to the water resources. Management of water resources, both in terms of quality and quantity therefore requires an understanding of the interaction between water, the environment and man. Without such an understanding, management of water resources will be piecemeal. In carrying out this, there is a need to monitor supply and demand of the resource, consider demographic factor and consider conjunctive use of surface and ground water resources. (Jimoh, 2010).

There are various factors which cause the use of water to vary. These factors are; pattern and standard of living, quality of water, cultural habits, whether the water is charged for, the cost and the purpose of use (Middle, 1978). Water use and consumption data are expressed in litres per capita per day (lpcd). A network of pipes is used to distribute water to a community. There are various ways in which this can be done e.g. a branching pattern with dead ends and a grid patterns with a large diameter pipe loop around an area with a high demand for water. Often a combination of the two is used in a city.

Water distribution networks serve many other purposes besides the provision of water for human consumption. Piped water is used for washing, sanitation, irrigation and fire fighting. Networks are designed to meet peak demands; in parts of the network this creates low-flow conditions that can contribute to the deterioration of microbial and chemical water quality. To maintain microbial quality, the network should be designed and operated to prevent entrance of contaminants, to maintain disinfectant residual concentrations within a locally predetermined range and to minimize the transit time (or age of the water after leaving the treatment works).

Depending upon the topography, the location of the source and other considerations, water can be transported to a community in a numbers of ways; viz canals, flumes, tunnels and pressure pipes, using gravity or pressurized systems as the topography allows.

1.2 Statement of the Problem

Bida Town is undergoing rapid expansion and it lacks adequate supply of potable water which has given room to the use of different water sources like water wells, shallow boreholes and even water from streams. This sometimes results in outbreaks of water borne diseases. According to 1991

Census, Bida had a population of 172,898 people while in 2006, the population was 185,553 (NPC, 2006) indicating a growth rate of 2.5%.

Some of the problems affecting supply in Bida town include; Pipe leakages, short supply of water treatment chemicals, ineffective means of communication, erratic power supply, bad habit of water use, inadequate provision of service reservoirs and breakdown of equipments. (ADB, 1997).

Bida water is supplied from Mana Water Works which has a designed capacity of 27,000 m^3/day commissioned in the year 1986 for 150,000 population equivalent. This project was executed by Biwater Company of United Kingdom. The total design capacity of the plants is inadequate compared to the required amount of 62,035 m^3/day (for a population of 185,553) leaving a deficit of 35,035 m^3/day. There is also inability of water works to run all the duty pumps for optimal production, comparatively affecting output. In general, the facilities have been overstretched. More areas are expanded and the distribution system has not been extended to these areas.

1.3 Aim and Objectives

The study is aimed at analyzing the distribution network of Bida Water Supply Scheme and the objectives are to:-

i) appraise the existing distribution network.

ii) analyse the present water distribution network using Epanet software and compare with the optimum case

iii) analyse the system under future scenario

1.4 Significance of study

Provision of water is perhaps the important among all the municipal services, people depends on water for drinking, washing, cooking and other domestic needs. Water supply and distribution must therefore meet the requirements for public, commercial and industrial activities both in quality and quantity.

This project is worth embarking upon for the following reasons:

i) It will enhance the update of comprehensive master plan for supply, transmission, and distribution facilities to accommodate growth of the city.

ii) It will provide for replacement of water supply infrastructure to maintain current capacity as it becomes obsolete.

1.5 Scope of the Research

The scope of the study includes analysis of all the reservoirs and storage tanks, pump/booster stations, and the pipes (primary and secondary lines).

1.6 Limitations

Epanet software used for this analysis performs a demand driven hydraulic simulation; which assumed that water demand is always satisfied for the customer by assuming that all pipes are full at all times. This assumption is correct only for the network with relatively high pressures. Actual discharge of water at individual node depends on pressure at that node. The software is therefore a pressure driven methodology type which consists of solving modified hydraulic equilibrium equations where water discharges are among other unknowns. (Guidolin, M., Burovsky, Z., Kapelan, Z., and Savic, D.A., 2010).

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 Background

During the pre independence era in Nigeria, water supply had been on human and animal carriage from rivers, streams, ponds, spring and wells. Between 1990 and 1940 some schemes were designed and constructed to supply hospitals and government establishments. However, as far back as 1960 the Federal Government of Nigeria became convinced of the necessity of adequate management of water supply scheme. In 1964, the Nigeria Water Corporation Law emerged which led to the establishment of Water Corporations between 1974 and 1976 with Bida Water Works in Niger State as one of them`. With the rising trend of the population and increasing demand of water, further expansion of the water works was carried out to complement the existing one. This led to the construction of another intake room and a reservoir of one million litres capacity to augment the former half a million litres capacity of water. Yet, this could not balance the quantity of water demanded and the water quality supplied. (Ibrahim and Nmadu, 2012)

The source of water in Bida is River Gbako which flows from River Chanchaga in Minna and their tributaries such River Musa, Chiken and Edoko to the water works station located at Manna village (about 10 km away to the east of Bida). The raw water is collected through a structure known as the intake room for treatment and processing before final distribution to the consumers.

Water distribution is the process of carrying water to consumers. It takes different method around the world from pressurized municipal water that convey water directly into individual homes to travelling tanker trucks that distribute water to community access points. Water distribution is not just about getting supply of water to people who need it, but about efficient allocation and coverage. The process of water distributions starts with identifying a source of water and determining what form of treatment applicable to make it usable. The water is moved through treatment facilities into distribution systems, including network of pipes, canals and aqueducts.

Flows through network of pipes are a function of difference in pressure at the inlet or outlet points. A pipeline system is designed to carry a certain flow. The size of pipes must not be unnecessarily over-dimensioned and at the same time there should be sufficient pressure throughout the system.

2.1.1 Branched Network

Branching network with dead ends is divided into the trunk-line (primary feeder), main (secondary lines) and sub-mains (service lines). The main are connected to the trunks, while the sub-mains are connected to supply water to the buildings. In pipes with dead ends the flow of water is always in the same direction, and water is supplied to an area by a sample pipe (Middle, 1978)

The advantages of this network include:-

1. It is very simple method of water distribution.
2. Design of such pipe network is simple.
3. The required dimensions of the pipe are economical (Middle, 1978).

The disadvantages are:-

1. There would be stagnation of water at the dead ends of the network. There would be accumulation of sludge with time, this results in change of quality of water and can make the water not portable and consequently cause corrosion of the pipe.
2. Maintenance works in the network due to pipe burst, replacement or fixing of pipe appurtenances will result in cutting off water supply to downstream consumers. (Middle, 1978).

2.1.2 Loop Network

The system is mostly ideal for built-up areas, cities and towns where buildings are closely together (Jeppson, 1977)

The advantages of loop network include:-

1. The sources could be more than one
2. Even distribution of pressure.
3. Supply becomes mostly reliable.
4. It has even distribution of water.

The disadvantages include:-

1. The system is costlier.
2. It is more difficult to analyse. (Jeppson, 1977)

Generally, in all distribution works, the flow is not steady as the demand changes at all nodes, thus the design is normally made for a maximum pressure requirement. Commonly used computer program for a pipe network is developed from Hardy cross method. The application of a system approach is by no means simple; one reason for this is that systems analysis lacks an accepted theoretical foundation (Quade and Boucher, 1968). System approaches vary, depending on the problem area and the analysis. In applying the linear theory method, it is not necessary to supply an initial estimate of the flow rate in each pipe. According to Wood and Charles (1972), a guess can be made even without satisfying continuity law. Further simplification is achieved assuming the estimate of the flow rate in each pipe to be unity (Jeppson, 1977).

2.2 Hydraulic of Water Distribution

The hydraulic equations, which describe pipe network system, are obtained from continuity and energy equations. For any good network design, basic equations must be satisfied. Hydraulic properties of the water and the characteristics of pipe network materials influence greatly the divide up of the flow.

$$Q = A \, x \, V \qquad (2.1)$$

where: Q = Flow rate (m^3 / sec)

A = Cross-sectional Area of the pipe (m^2)

V = Velocity of flow in pipe (m/s)

The Bernoulli’s equation states: - If the steady flow of frictionless, incompressible fluid along a streamline is considered, the total energy of a unit fluid mass would remain the same considering two sections. Figure 1.1 illustrate the Bernoulli distribution.

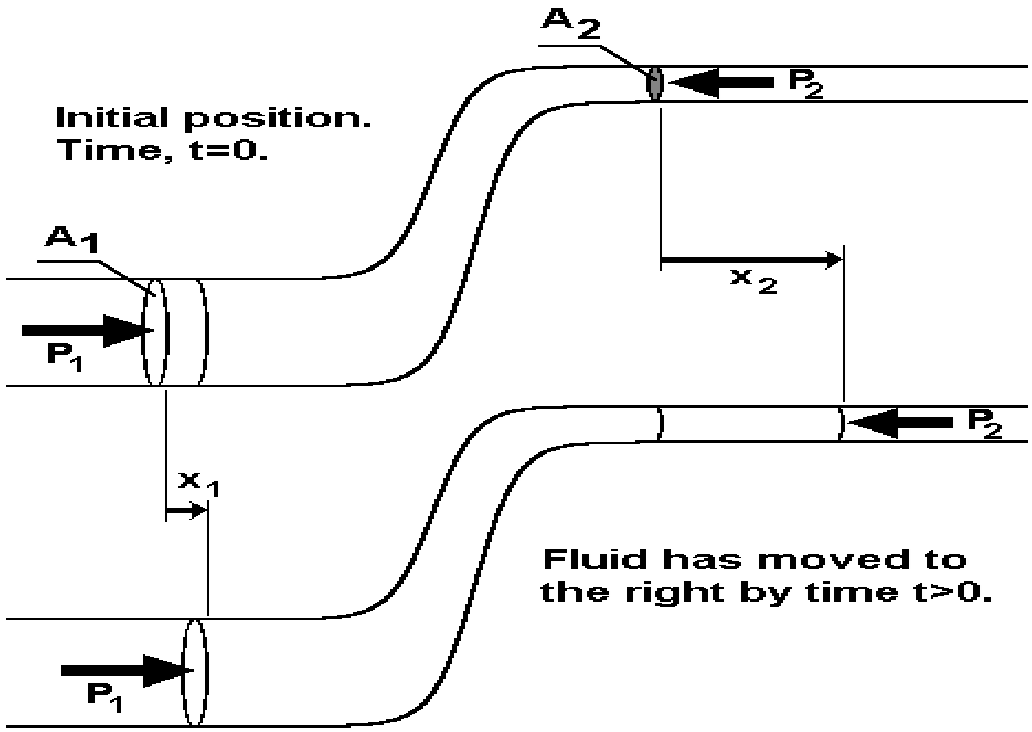


Figure 2.1 Bernoulli distribution of steady flow

$$P_1/\rho g + V_1^2/2g + X_1 = P_2/\rho g + V_2^2/2g + X_2 + h_f \qquad (2.2)$$

or

$$X_1 - X_2 + \frac{P_1 - P_2}{\rho g} + \frac{V_1^2 - V_2^2}{\rho g} - h_f = 0 \qquad (2.3)$$

where V = Mean velocity at section.

$V^2/2g$ = Velocity head (m) or the kinetic energy of the water

$P/\rho g$ = Pressure head (m)

X = The elevation head above a datum (m) or the potential energy

h_f = Head loss due to friction between the points of measurement.

This equation shows that it is the difference in potential energy, flow energy, and kinetic energy that actually has significance. The $X_1 - X_2$ is independent of the particular elevation datum, as it is the difference in elevation of the two points. Similarly, $P_1/\rho g - P_2/\rho g$ is the difference in pressure heads expressed in units of length of the fluid flowing and is not altered by the particular pressure datum selected. Since the velocity terms are not linear, their datum is fixed.

2.3 Methods of Analyzing Pipe Network

Many methods of pipe network analysis have been used in finding solution to pipe network problems. Some of these applications are discussed as follows:-

2.3.1 Hardy Cross Method

The oldest method of systematically solving the problem of steady flow in pipe network is hardy cross method (Cross, 1936).

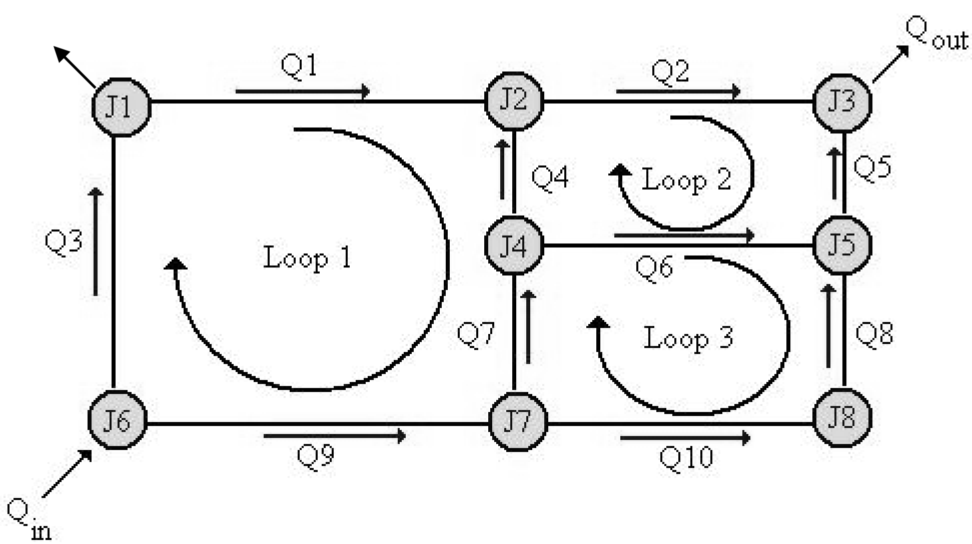


Figure 2.2 Hardy cross loop diagram

Hardy cross presented a method which described only a closed loop network without pump or reservoir and solving the equation by successive approximation of the flow to individually balance the equations. For unknown flow rate in the pipes, these are obtained by reasonably estimating initial flow rate which satisfy the continuity equation at each junction and computing the corrected flow rate in each pipe (Adeleke and Olawale, 2013). Each pipe estimated flow rate is then corrected through adding the corrected flow rate in the adjacent loop. The procedure is repeated until convergence is achieved and connections are negligible.

Hardy Cross Method is based on two principles, which are node law and loop law. The node law stated that the sum of flow towards any junction of node is zero (i.e. at any node including external flows, $\Sigma q = 0$) and loop law states that the sum of head losses around any closed loop is zero, i.e. $\Sigma h = 0$. Both approaches yield identical solutions but one is usually more expedient and less laborious depending on the type of problem. The theory developed by cross is as follows (for the method of balancing flows):

For any pipe in which Q_0 is an assumed initial discharge

$$Q_{ij} = Q_0 + \Delta Q \tag{2.4}$$

where $Q_{ij}$is the correct discharge, Q_0 is the initial approximation of flow, and Δq is the correction to be calculated. Then for each pipe,

$$h_f = rQ^n = r(Q_0 + \Delta Q)^n = r(Q_0^n + nQ_o^{n-1}\Delta Q + \cdots) \tag{2.5}$$

If ΔQ is small compared with Q_0, all terms of the series after the second may be dropped. Now for a circuit,

$$\sum h_f = \sum rQ\ |Q|^{n-1} = \sum rQ_0\ |Q_0|^{n-1} + \Delta Q \sum rn\ |Q_0|^{n-1} = 0 \tag{2.6}$$

in which ΔQ has been taken out of the summation because it is the same for all pipes in the circuit and absolute-value signs have been added to account for the direction of summation around the circuit. The last equation is solved for ΔQ in each circuit in the network.

$$\Delta Q = -\frac{\sum rQ_0\,|Q_0|^{n-1}}{\sum rn\,|Q_0|^{n-1}} \qquad (2.5)$$

where ΔQ is applied to each pipe in a circuit in accordance with Eq. 2.4, the directional sense is important; that is it adds to flows in the clockwise direction and subtracts from flows in the counter clockwise direction.

The advantages of Hardy Cross Method are: -

i) It can be analyzed manually without computer

ii) It is simple and easy in computation

iii) It is applicable to close loop

iv) It can analyze a common network of less than 50 pipes.

The disadvantages of Hardy Cross Method are:

i) It converges very slowly and therefore requires large number of iteration

ii) In some situation, the method starts diverging and does not converge at all

iii) The number of iteration increases as the size of the network increases

iv) The number of iterations requirement for convergence depend upon: -

   a) Convergence criterion

   b) The pipe resistance constants and

   c) The pipe discharges.

v) It analyses the network in parts, considers one loop or node at a time

vi) Effect of network size on the time per iteration increases gradually as the size of the network increases. (Mohammed, 2005)

2.3.2 Newton-Raphson Method

Hardy cross method attempt to solve the non-linear equations involved in network analysis by making certain assumption and high power correction are neglected. In Newton Raphson method, all the correction equations are solved simultaneously; convergence is achieved in a smaller number of iteration (Bhave, 1991). The non-linear equations in terms of unknown head at each pipe of the network were solved using the basic Newton Raphson techniques of successive iterative solution of non-linear equation. The method may be used to solve any of the three sets of equations which describe flow rate in pipe network, namely the equation considering the flow rate in each pipe as known, the head at each junction as unknown; or the corrective flow rate in and around each loop as unknown.

One of the best known procedures for finding the roots of an equation is the Newton-Raphson iteration method. This method is based on Taylor's theorem.

Let x_0 be a good estimate of r and let r = $x_0 + h$. Since the true root is r, and $h = r - x_0$, the number h measures how far the estimate x_0 is from the truth.

Since h is 'small' we can use the linear (tangent line) approximation to conclude that

$0 = f(r) = f(x_0 + h) \approx f(x_0) + h\,f'(x_0)$,

And therefore, unless $f'(x_0)$ is close to 0,

$$h \approx -\frac{f(X_0)}{f\prime(X_0)} \quad (2.6)$$

It follows that

$$r = x_0 + h \approx x_0 - \frac{f(X_0)}{f\prime(X_0)} \quad (2.7)$$

The new improved estimate, x_1 of r is therefore given by

$$x_1 = x_0 - \frac{f(X_0)}{f\prime(X_0)}. \quad (2.8)$$

The next estimate x_2 is obtained from x_1 in exactly the same way as x_1 was obtained from x_0:

$$x_2 = x_1 - \frac{f(X_1)}{f\prime(X_1)}. \quad (2.9)$$

Generally, if x_n is the current estimate, then the next estimate x_{n+1} is given by

$$x_{n+1} = x_n - \frac{f(x_n)}{f'(x_n)} \quad (2.10)$$

The advantages of Newton-Raphson Method are:

i. It has a high computational efficiency
ii. It has quadratic convergence
iii. It requires less iteration to obtain the solution with a given precision than if linear convergence occurs
iv. It is easily implemented in a computer algorithm
v. It requires less computer storage for a given number of equations

vi. It analyses the whole (all loops) simultaneously.

The disadvantages of Newton-Raphson Method are:

i. It requires a reasonable accurate initialization before it converges (i.e. it has convergence problem)

ii. It cannot be analyzed manually without computer. (Mohammed, 2005)

2.3.3 Linear Theory Method

This method transforms the non-linear loop equation into the linear form and matrix method is used in solving the system of equation which considers the flow rate as unknown. The method has distinct advantages over the Newton Raphson or Hardy Cross methods: First it does not require an initialization and secondly it always converges in a relatively little iteration (Wood and Charles, 1972). It is a very balance method whereby all the equations are solved simultaneously.

Each nonlinear head loss term is linearised using the first two terms in Taylor series:

$$f(Q) = f(q) + \frac{\partial f}{\partial q}(Q - q) \tag{2.11}$$

where q = flow rate from previous iteration, and

Q = unknown flow rate

So,

$$KQ^n = Kq^n + nKq^{n-1}(Q - q)$$

$$= DQ + D' \tag{2.12}$$

where

$D = nKq^{n-1}$ and $D' = (1 - n)Kq^n$

For loops with pumps, nonlinear pump characteristic equation (LHS) is replaced with linear equation (RHS):

$$AQ^2 + BQ + H = Aq^2 + Bq + H + (2Aq + B)(Q - q)$$

$$= EQ + E' \qquad (2.13)$$

where

$E = 2Aq + B$, and

$E' = H - Aq^2$

The advantages of Linear Theory Method are: -

i. It does not require the supply of an initial flow rate estimate in each of the pipe; a guess can be made without satisfying continuity law.

ii. The method is more simplified if the flow rate is assumed to be unity (or unknown) in each pipe.

The Disadvantages of Linear Method

i. It is used in solving the head oriented equations but the corrective loop equation is not recommended.

ii. It can be analysed manually without a computer. (Mohammed, 2005)

2.4 Computer Software for Pipe Network Analysis

2.4.1 Water distribution modeling using WaterCAD

WaterCAD is a solution for water quality modeling of water distribution systems. It features advanced interoperability, model building, optimization, and asset management tools. Engineers can use its built-in water-quality components to perform constituent, water-age, tank-mixing and source-trace analysis to develop comprehensive chlorination schedules, simulate mock contamination events, model flow-paced and mass-booster stations, and visualize zones of influence for every water source (Bentley, 2012).

WaterCAD Features include the following: -

- Easier model maintenance with Geographic Information System identity property: The new GIS-ID property can be used for sustaining associations between records in source file and elements in the model.
- Model Builder advancements such as the ability to import a subset of data using queries. This allows users to refine the data to import only the needed information, without having to distort the data from its original format.
- Ability to create Google Earth files for results display (Bentley, 2012).

WaterCAD is a robust and easy-to-use modeling program for water distribution that can be customised with additional modeling platforms. WaterCAD can run in two interoperable platforms, and allow the designer to select the environment that is better fit for the modeling requirements.

1. Windows stand-alone: Enjoy unparalleled ease-of-use and versality, multiple background support, excellent thematic mapping, effective element symbology features, and conversion utilities from CAD, GIS, and databases.

2. Micro Station: The ability to run WaterCAD from within Micro Station is included at no extra cost. Seamlessly use every WaterCAD feature within Micro Station's powerful engineering design and geospatial environment (Bentley, 2012).

WaterCAD is not used for this analysis because it is not s free software and cannot be freely copied and distributed.

2.4.2 Pipe network systems analysis using EPANET computer software

EPANET is a computer program that performs extended period simulation of hydraulic and water quality behavior within pressurized pipe networks. A network consists of pipes, nodes (pipe junctions), pumps, valves and storage tanks or reservoirs. EPANET tracks the flow of water in each pipe, the pressure at each node, the height of water in each tank, and the concentration of a chemical species throughout the network during a simulation period comprised of multiple time steps. In addition to chemical species, water age and source tracing can also be simulated (Rossman, 2000).

EPANET was developed by the Water Supply and Water Resources Division of the U.S Environmental Protection Agency, National Risk Management Research Laboratory. It was designed to be a research tool for improving understanding of the movement and fate of drinking water constituents within distribution systems. It can be used for many different kinds of applications in distribution systems analysis. Sampling program design, hydraulic model calibration, chlorine residual analysis, and consumer exposure assessment are some examples. EPANET can help assess alternative management strategies for improving water quality throughout a system.

Running under Windows environment, EPANET provides an integrated environment for editing network input data, running hydraulic and water quality simulations, and viewing the results in a variety of formats. These include color-coded network maps, data tables, time series graphs, and contour plots.

The advantages of Epanet systems analysis are as follows: -

1. Places no limit on the size of the network that can be analyzed
2. Computes friction head loss using the Hazen-Williams, Darcy-Weisbach, or Chezy-Manning formulas
3. Includes minor head losses for bends and fittings
4. Models constant or variable speed pumps
5. Computes pumping energy and cost
6. Models various types of valves including shutoff, check, pressure regulating, and flow control valves
7. Allows storage tanks to have any shape (i.e., diameter can vary with height)
8. Considers multiple demand categories at nodes, each with its own pattern of time variation
9. Models pressure-dependent flow issuing from emitters (sprinkler heads)
10. Can base system operation on both simple tank level or timer controls and on complex rule-based control. (Rossman, 2000).

EPANET is used for this analysis because it is public domain software that may be freely copied and distributed.

2.5 Physical Components of the Network

EPANET models a water distribution system as a collection of links connected to nodes. The links represent pipes, pumps, and control valves. The nodes represent junctions, tanks, and reservoirs (Rossman, 2000). Figure 2.2 illustrates how these objects can be connected to one another to form a network.

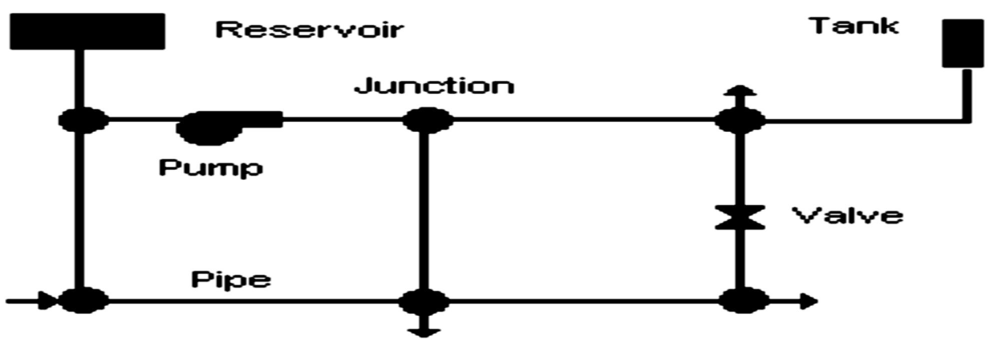


Figure 2.3 Physical Components in a Water Distribution System

1. Junctions

Junctions are points in the network where links join together and where water enters or leaves the network. The basic input data required for junctions are the elevation above some reference (usually mean sea level) and water demand (rate of withdrawal from the network). The output results computed for junctions at all time periods of a simulation are the hydraulic head (internal energy per unit weight of fluid) and pressure

Junctions can also have their demand vary with time; it can have multiple categories of demands assigned to them and could have negative demands indicating that water is entering the network. Junction can also contain emitters (or sprinklers) which make the outflow rate depend on the pressure. (Rossman, 2000).

2. **Reservoirs**

Reservoirs are nodes that represent an infinite external source or sink of water to the network. They are used to model such things as lakes, rivers, groundwater aquifers, and tie-ins to other systems. Reservoirs can also serve as water quality source points.

The primary input properties for a reservoir are its hydraulic head (equal to the water surface elevation if the reservoir is not under pressure) and its initial quality for water quality analysis. Because a reservoir is a boundary point to a network, its head and water quality cannot be affected by what happens within the network. Therefore it has no computed output properties. However its head can be made to vary with time by assigning a time pattern to it. (Rossman, 2000).

3. **Tanks**

Tanks are nodes with storage capacity, where the volume of stored water can vary with time during a simulation. The primary input properties for tanks are bottom elevation (where water level is zero) and initial, minimum and maximum water levels. The principal outputs computed over time are the hydraulic head (water surface elevation).

Tanks are required to operate within their minimum and maximum levels. EPANET stops outflow if a tank is at its minimum level and stops inflow if it is at its maximum level. Tanks can also serve as water quality source points. (Rossman, 2000).

4. **Pipes**

Pipes are links that convey water from one point in the network to another. EPANET assumes that all pipes are full at all times. Flow direction is from the end at higher hydraulic head (internal energy per weight of water) to that at lower head. The principal hydraulic input parameters for pipes are the start and end nodes, the diameter, length, roughness coefficient (for determining headloss), status (open, closed, or contains a check valve). The status parameter allows pipes to implicitly contain shutoff (gate) valves and check (non-return) valves which allow flow in only one direction. (Rossman, 2000).

The water quality inputs for pipes consist of bulk reaction coefficient and wall reaction coefficient. While the computed outputs for pipes include the flow rate**,** velocity**,** headloss**,** Darcy-Weisbach friction factor, average reaction rate (over the pipe length) and average water quality (over the pipe length). The hydraulic head lost by water flowing in a pipe due to friction with the pipe walls can be computed using either Hazen-Williams formula, Darcy-Weisbach formula or Chezy-Manning formula.

With the Darcy-Weisbach formula, EPANET uses different methods to compute the friction factor f, depending on the flow regime:

- The Hagen–Poiseuille formula is used for laminar flow ($Re < 2,000$).

- The Swamee and Jain approximation to the Colebrook-White equation is used for fully turbulent flow (Re > 4,000).
- A cubic interpolation from the Moody Diagram is used for transitional flow (2,000 < Re < 4,000). (Rossman, 2000).

Table 3.1 lists expressions for the resistance coefficient and values for the flow exponent for each of the formulas. Each formula uses a different pipe roughness coefficient that must be determined empirically. Table 3.2 lists general ranges of these coefficients for different types of new pipe materials. Note that a pipe's roughness coefficient can change considerably with age.

Table 3.1: Pipe Headloss Formulae for Full Flow

Formula	***Resistance Coefficient (A)***	***Flow Exponent (B)***
Hazen-Williams	$4.727\, C - 1.852\, d - 4.871\, L$	1.852
Darcy-Weisbach	$0.0252\, f(e, d, q) d - 5L$	2
Chezy-Manning	$4.66\, n2\, d - 5.33\, L$	2

Notes:

C = Hazen-Williams roughness coefficient

ε = Darcy-Weisbach roughness coefficient (ft)

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

n = Manning roughness coefficient

d = pipe diameter (ft)

L = pipe length (ft)

q = flow rate (cfs) (Rossman, 2000).

Table 3.2: Roughness Coefficients for New Pipes

Material	***Hazen-Williams C*** ***(unitless)***	***Darcy-Weisbach* e** ***(feet x 10^{-3})***	***Manning's n*** ***(unitless)***
Cast-Iron	130 – 140	0.85	0.012 - 0.015
Concrete or Concrete Lined	120 – 140	1.0 - 10	0.012 - 0.017
Galvanized Iron	120	0.5	0.015 - 0.017
Plastic	140 – 150	0.005	0.011 - 0.015
Steel	140 – 150	0.15	0.015 - 0.017

Vitrified Clay	110	0.13 - 0.015

5. Pumps

Pumps are the links that convey energy to a fluid thereby increasing its hydraulic head. The primary input parameters for a pump are its start and end nodes and the pump curve i.e. the combination of heads and flows that the pump can provide. In lieu of a pump curve, the pump could be designated as a constant energy device, one that supplies a stable amount of energy (horsepower or kilowatts) to the fluid for all combinations of flow and head. The primary output parameters are head gain flow. Flow through a pump is unidirectional and EPANET does not permit a pump to operate outside the range of its pump curve. Variable speed pumps could also be considered by setting the speed to change under these same types of conditions. In essence, the original pump curve supplied to the program has a relative speed setting of 1. If the speed doubles, then the relative setting would be 2; if run at half speed, the relative setting is 0.5 and so on. Adjusting the pump speed therefore, shifts the position and shape of the pump curve.

Pumps can be turned on and off at preset times or when some certain conditions exist in the network. The operation of a pump can also be described by assigning a time pattern of relative speed settings to it. EPANET also has the ability of computing the energy consumption and cost of a pump. Each pump can be assigned an efficiency curve and schedule of energy prices. If these are not supplied then a set of global energy options will be used.

Since the flow is unidirectional, if system conditions require more head than the pump can deliver, EPANET shuts the pump off. If more than the maximum flow is needed, EPANET projects the pump curve to the required flow, even if this produces a negative head. In either cases a warning message will be issued. (Rossman, 2000).

6. **Valves**

Valves are links that limit the flow or pressure at a specific point in the network. Their primary input parameters are the start and end nodes, diameter, setting and status. The computed outputs are headloss and flow rate. (Rossman, 2000).

2.6 Applications of EPANET in Analysis of Distribution problems in Nigeria

In 2013, Saminu, Abubakar, Nasiru and Sagir worked on the Design of Nigerian Defence Academy Water Distribution Network Using EPANET. The project area is the NDA permanent site Kaduna, which is situated along Mando road, Afaka Kaduna and Lagos expressway with approximated total population of 6,935 persons. The distribution network consists of 63 pipes of different materials, 57 junctions, 3 tanks and 2 source reservoir from which water is pumped to the surface reservoir and later on distributed to the network. At the end of the analysis, it was found that the resulting pressures at all the junctions and the flows with their velocities at all pipes are adequate enough to provide water to the study area". (Saminu et al., 2013)

CHAPTER THREE

3.0 MATERIALS AND METHOD

3.1 Description of the Study Area

Bida is one of the largest city in Niger State, it geographical coordinates are latitude 9˚05′N to 6˚01′E and longitude 9.083˚N to 6.017˚E occupying an area of 51 km^2. It is a dry, arid town located south west of Minna, Capital of Niger State. The area is of low to moderate relief with a few scattered laterite-capped hills where elevations rarely exceed 400m above sea level. It is drained by River Niger, and its major tributaries are Kaduna, Gbako, Gurara, Chanchaga, Kampe, to mention the major perennial streams.

The principal economic activity in this city is agriculture and these rivers play a great role here as they supply water not only for irrigation and hydroelectricity but also their flood plains are cultivated. Vegetation is of the guinea savannah type and rainfall ranges from about 1,000mm to 1,500mm with a marked rainy season from April to October, the maximum being in July and August. Annual average air temperature lies at about 27˚ Celsius.

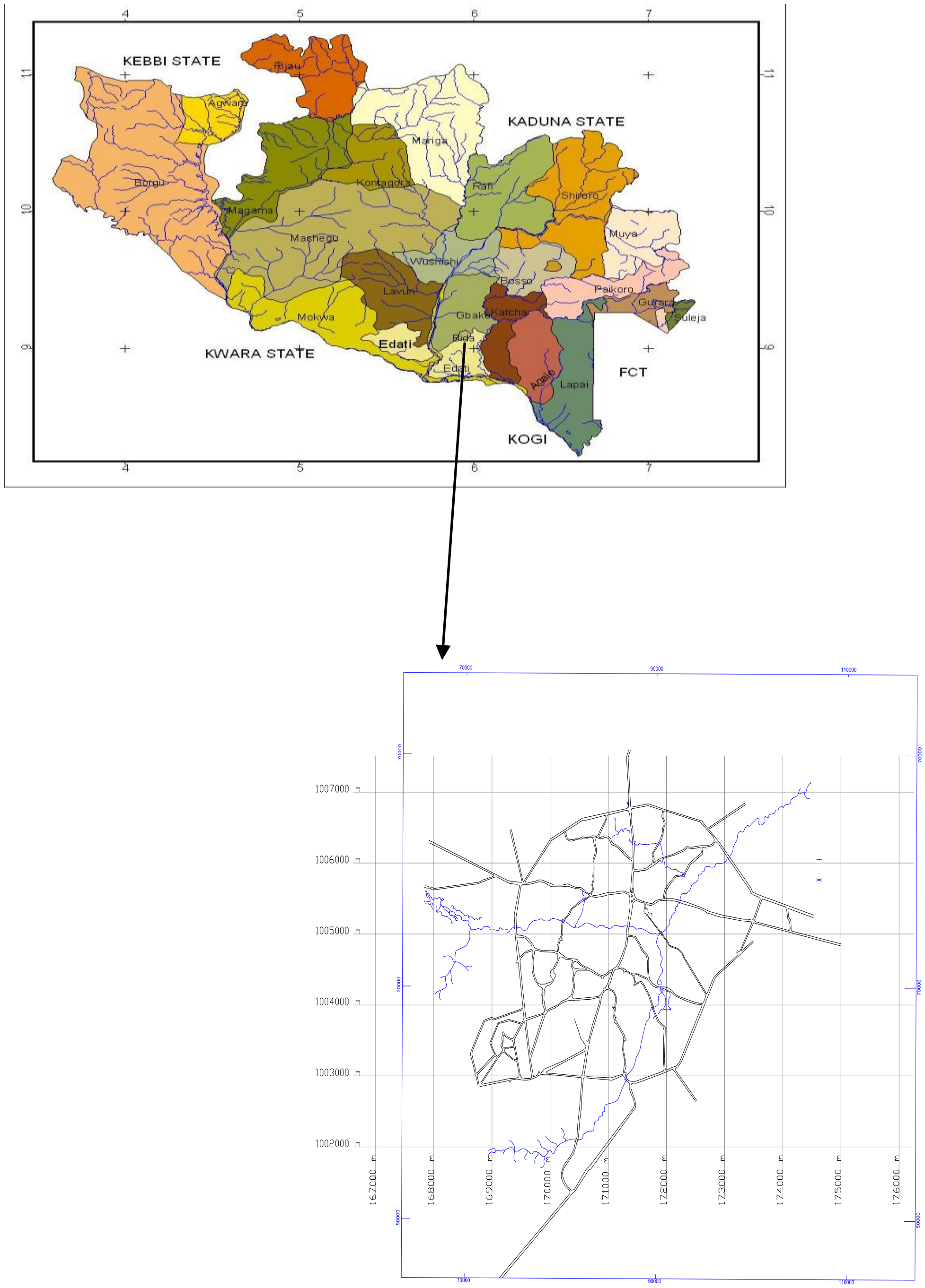


Figure 3.1: Location of the study area

Source: (Federal Survey, 1976)

3.2 Data Collation

The following data were collated for this study:

i) Population Data of Bida obtained from 2006 Population census

ii) Population Data of the loop based on Town Development Committee assessment

iii) Water Supply Guide Map showing the Distribution Network obtained from Niger State Water Board (NSWB) as shown in figure 3.2

iv) Data on the capacities of reservoirs, and storage capacities from NSWB

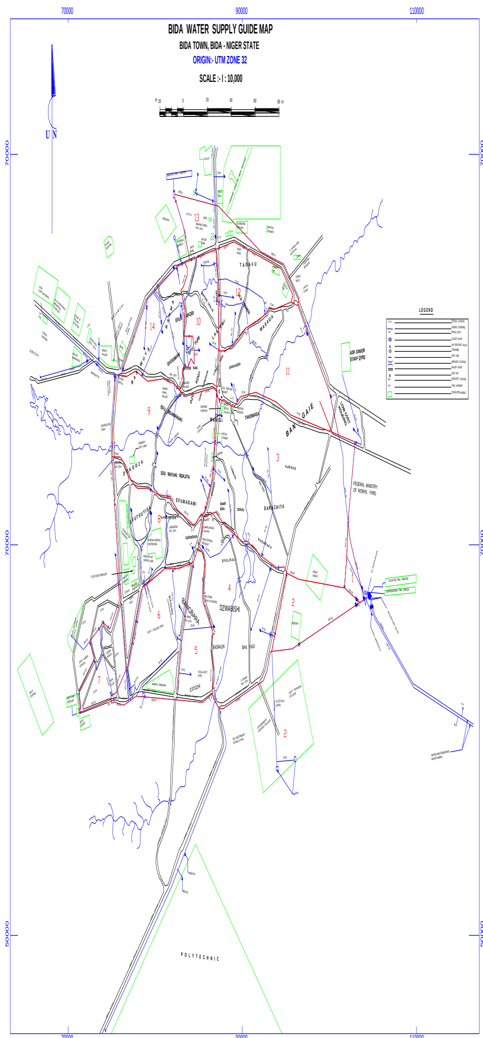


Figure 3.2 Bida Water Supply Guide Map

Source: (Federal Survey, 1976)

3.3 Population Data and Water Demand

An accurate population data of the town is necessary to determine water requirement of the town. The population data, for the purpose of a water system, include all persons who will depend upon it for their drinking water, patients in the health posts, students living in dormitories, and employees in government offices. It is imperative to take into consideration the needs for the present and future population of the area to be served with water.

The area under review is undergoing rapid development; therefore the initial phase of population growth will be rapid. In view of this, the compound method of population forecasting was considered. The future population was estimated using equation 3.1

$$P_n = P_i(1 + r)^n \quad (3.1)$$

where P_n = the projected population for the year, n

P_i = initial Population

r = population growth rate

n = design period in year n

Water demand fluctuates both in dry and raining season, the design was based on dry season demand which is higher than the wet season. The amount of domestic water consumption per person varies according to the living condition of the consumers; average daily consumption of 120 litres per day (National Water Supply and Sanitation Policy 2000) is used for this project. The total water demand generally amounts to 50 to 55% of the domestic water consumption (Chatterjee, 1987).

Average daily demand = population x per capita consumption

Peak daily demand = 2 x Daily average demand

3.4 Procedure of analysis

The following steps were used in analyzing the Bida water distribution system

1. Network was digitalized using AutoCAD
2. The principal characteristic of the network placed in a text was imported from AutoCAD. EPANET is capable of importing a geometric description of a pipe network in a simple text format. This description simply contains the ID labels and map coordinates of the nodes and the ID labels and end nodes of the links. This simplifies the process of using other programs, such as CAD and GIS packages, to digitize network geometric data and then transfer these data to EPANET. (Rossman, 2000).
3. The properties of the objects that make up the system were edited. The properties of objects that appear on the Network Map are edited through Property Editor. These properties include Junctions, Reservoirs, Tanks, Pumps, Pipes, Valves, or Labels. To edit any of these objects, select the object on the map or from the Data Browser, then click the edit button on the Data Browser, or simply double-click the object on the map. The basic Epanet workspace is pictured in Figure 3.3. It is consists of the following user interface elements: a Menu Bar, a Status Bar, two Toolbars, the Network Map window, a Browser window and a Property Editor window. (Rossman, 2000).

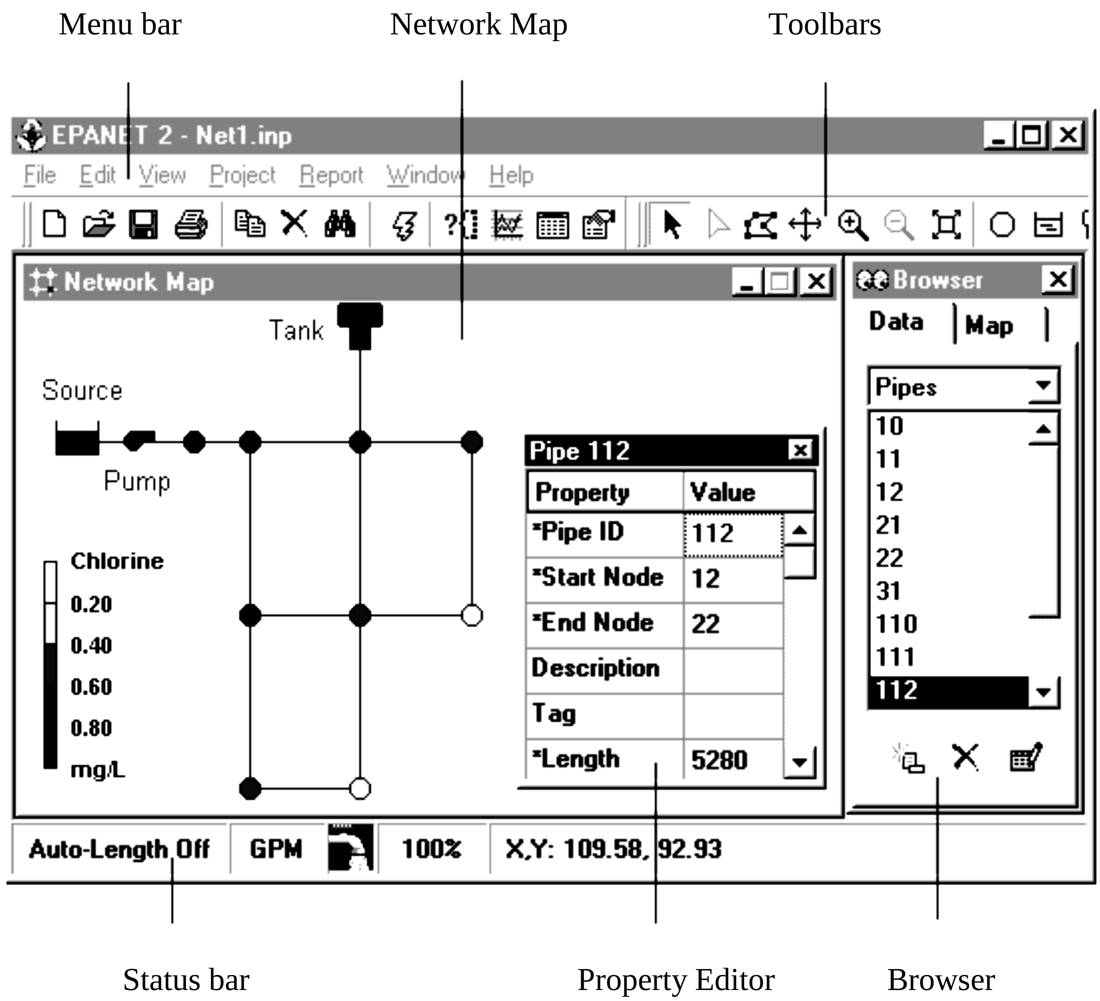


Figure 3.3 Epanet's workspace

4. The system operation was then described. Time Patterns, Curves, and Controls have special editors that are used to define their properties. To edit one of these objects, the object is selected from the Data Browser and then clicks the Edit button. In addition, the Property Editor for Junctions contains an ellipsis button in the field for Demand Categories that brings up a special Demand Editor when clicked. Similarly, the Source Quality field in the Property Editor for Junctions, Reservoirs, and Tanks has a button that launches a special Source Quality editor. Select a set of analysis options

5. Hydraulic analysis was executed based on the following steps:

   a. Select **Project** and then **Run Analysis** or simply click the icon on the Standard Toolbar.

   b. The progress of the analysis will be shown in a Run Status window.

   c. Click on **OK** when the analysis ends.

   If the analysis runs successfully the icon will appear in the Run Status section of the Status Bar at the bottom of the EPANET workspace. Any error or warning messages will appear in a Status Report window. After a successful run has been made, the properties of the network was edited and the faucet icon changes to a broken faucet indicating that the current computed results no longer apply to the modified network.

6. Results of the analysis. The database values and results of a simulation can be viewed directly on the Network Map through different ways: For the current settings on the Map Browser, the nodes and links of the map is colored according to the color coding used in the Map Legends. The colour of the map is updated as a new time period is selected in the Browser. When the Flyover Map Labeling program preference is selected, moving the mouse over any node or link will display its ID label and the value of the current viewing parameter for that node or link in a hint-style box. ID labels and viewing parameter values can be displayed next to all nodes and/or links by selecting the appropriate options on the Notation page of the Map Options dialog form (Rossman, 2000).

7. The analysis was carried out for the existing water distribution system of Bida Town.
8. The optimum water distribution systems on the respective present and future water demand scenario was subsequently obtained.

CHAPTER FOUR

4.0 RESULTS AND DISCUSSION

4.1 Presentation of Results

4.1.1 System Layout

The water distribution network of Bida is presented in Figure 4.1. The intake to the system is from Manna Water Works and the reference node indicated by letter R-1 is the location of the reservoir. The network has 14 loops and 27 nodes.

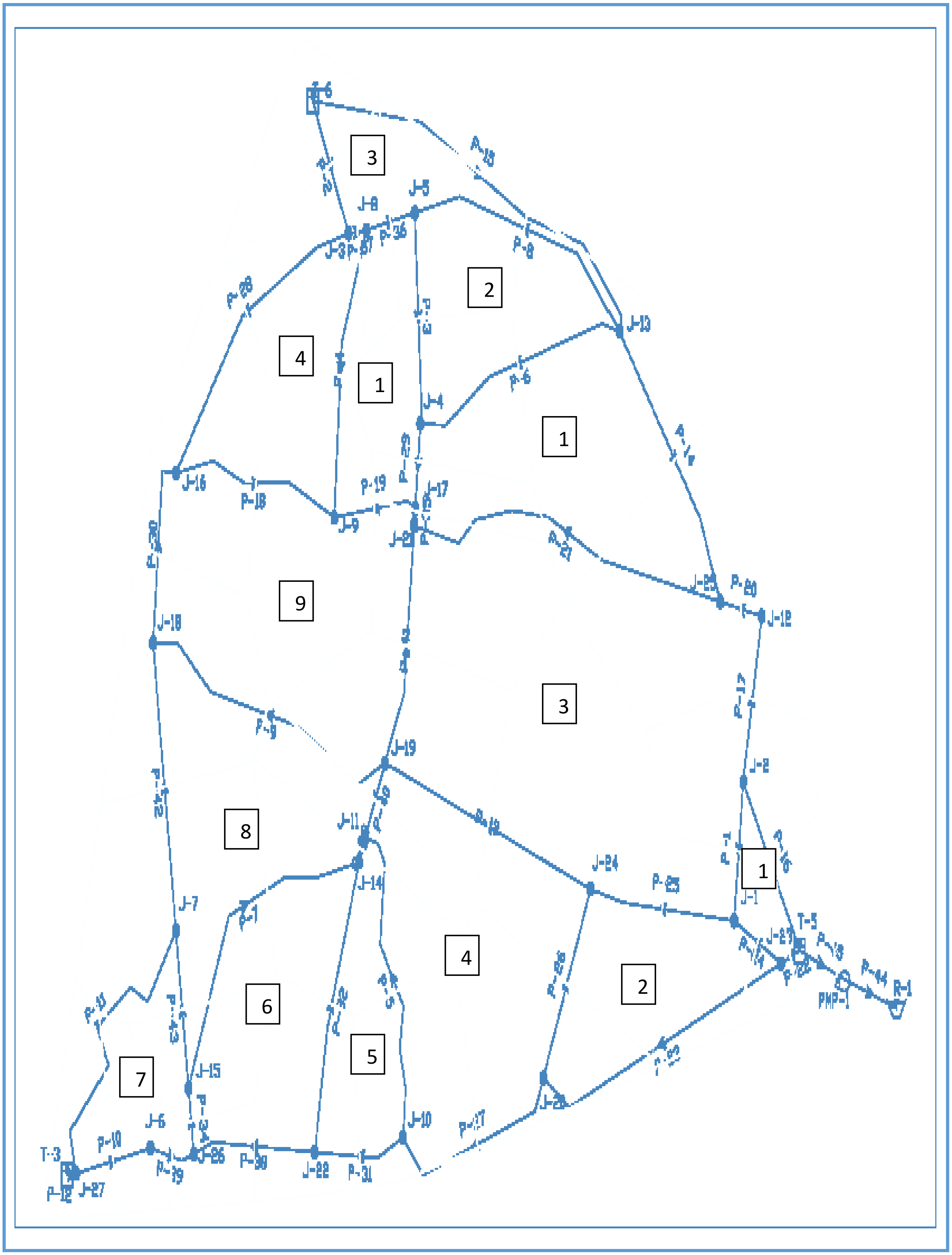


Figure 4.1 **System Layout of the Network**

4.1.2 Estimated water demand

The population based on the census exercise in 2006 (NPC, 2006) was 185,553. The population distribution of the area in the sectors with reference to the network loops was collected from the town development committee. Water consumption per meter length of pipe for each loop is dependent on the population per loop, average daily consumption per day. This implies consumption is affected by total length of pipes in a loop. Domestic water demand for the loop involves the water required for drinking, cooking, bathing, cleaning, gardening, and sanitary purposes. Table 4.1 shows the population distribution and water demand for each loop. Loop 3 is the largest loop and has the maximum demand of 0.1 m^3/s, while loop 1 which is the smallest loop has the minimum demand of 0.008. The total water demand is 0.718 m^3/s which is equal to 62,035.20 m^3/day as compared to 27,000 m^3/day used as the design capacity for a population of 150,000 population equivalent during the construction in 1986.

TABLE 4.1: Area and Population Distribution in the Loop

Loop No.	Area of the Loop (hecter)	Population per Loop (2006)	Density Population/Area	Water demand for the loop (m^3/s)
1.	16.4	1,767	107.74	0.008
2.	80.7	3,534	43.79	0.014
3.	327.8	24,740	75.47	0.100
4.	169.3	21,206	125.26	0.084
5.	55.2	8,836	160.07	0.014
6.	100.2	14,137	141.09	0.056
7.	49.8	5,302	106.47	0.020
8.	125	19,439	155.51	0.078
9.	174.6	22,973	131.58	0.092
10.	63.6	10,603	166.71	0.042
11.	138.7	17,673	127.42	0.072

12.	82.1	15,905	193.73	0.060
13.	48.4	7,069	146.05	0.028
14.	83.8	12,370	147.61	0.050
TOTAL	**1,515.60**	**185,553**	**1,828.50**	**0.718**

Spot heights of the junction were determined through the existing distribution network and field measurement. This was done to determine the height and the distances of various lines as shown in the Table 4.2. There are 7 categories of pipes in the system. The diameters vary from 75 mm to 450 mm (Table 4.3). The main line use 300 mm to 450 mm diameter pipes while the secondary and tertiary lines use 75 mm to 250 mm diameter pipes. There are 4 distribution reservoirs in the system; the location and elevation of the reservoirs are shown in Table 4.4.

TABLE 4.2: Elevation of Nodes

Node no.	Elevation (m)	Node no.	Elevation (m)

1	154.18	15	184.82
2	146.35	16	149.76
3	149.59	17	128.63
4	133.12	18	136.31
5	140.74	19	136.83
6	187.63	20	146.66
7	171.00	21	127.33
8	146.76	22	158.52
9	133.53	23	162.82
10	136.91	24	137.52
11	144.91	25	134.99
12	138.04	26	187.38
13	112.68	27	187.39
14	147.00		

TABLE 4.3: **Categories of pipe**

No.	Pipe diameter (mm)	Length (km)
1	450	1.48
2	300	10.97
3	250	8.18
4	200	1.45
5	150	11.72
6	100	3.07
7	75	5.50
	Total Length of pipe	**42.37**

TABLE 4.4: **Characteristics of distribution**

S/N	Type	Capacity (m^3)	Elevation (m)

1	High Level Elevated Costain Steel Tank	4,500	191.55
2	Under Ground Concrete Tank	900	131.26
3	Biwater Elevated Steel Tank	2000	173.32
4	G.R.A New Tank	1000	171.42

4.1.3 Hydraulic Simulation of Pipe Network

Based on the present population density, demands were assigned to individual nodes. The Hazen – William friction co-efficient used for each pipe in the network is 130 (Ductile iron). The existing network was analyzed with all its original parameters and tested, the various simulation output obtained are shown on Tables 4.5-4.8.

Table 4.5 shows the input results for the link which indicate the direction of flow for each of the node showing the start and end nodes as well as the length and diameter of each of the links. Table 4.6 shows the output of the energy consumption by each of the installed pumps.

TABLE 4.5: Input results for the link

Link ID	Start Node	End Node	Length m	Diameter mm
P-1	J-2	J-1	564	150
P-2	T-6	J-3	598	200
P-3	J-5	J-4	851	200
P-4	J-8	J-9	1197	75
P-5	J-11	J-10	1365	100
P-6	J-13	J-4	1584	75
P-8	J-5	J-13	1706	100
P-9	J-18	J-19	1906	150
P-10	J-6	J-27	570	150
P-11	J-27	J-7	1548	150

P-12	J-27	T-3	69	150	
P-13	J-19	J-24	1629	250	
P-14	J-13	J-25	1336	300	
P-15	T-6	J-13	2618	300	
P-16	T-5	J-2	802	450	
P-17	J-2	J-12	682	450	
P-18	J-16	J-9	1243	150	
P-19	J-9	J-17	620	150	
P-20	J-12	J-25	319	75	
P-21	J-21	J-25	2400	75	
P-22	T-5	J-23	149	300	
P-23	J-23	J-20	1926	300	
P-24	J-23	J-1	394	250	
P-25	J-1	J-24	1090	250	
P-26	J-24	J-20	840	300	
P-27	J-20	J-10	1243	300	
P-28	J-3	J-16	1677	250	
P-29	J-4	J-17	343	150	
P-30	J-18	J-16	798	250	
P-31	J-10	J-22	682	300	
P-32	J-14	J-22	1218	150	
P-33	J-14	J-11	110	150	
P-34	J-26	J-15	266	250	
P-35	J-17	J-21	72	150	
P-36	J-8	J-5	370	250	
P-37	J-3	J-8	136	250	
P-38	J-22	J-26	921	300	
P-39	J-26	J-6	335	300	
P-40	J-11	J-19	336	150	
P-41	J-19	J-21	994	150	
P-42	J-7	J-18	1172	250	
P-43	J-15	J-7	644	250	
P-44	R-1	182-A	0.001	1000	
P-45	182-B	T-5	352	150	
P-58	203-B	J-14	1014	150	
P-59	J-15	203-A	759	150	
P-61	J-22	204-A	767	300	
P-62	204-B	J-26	155	300	
PMP-1	182-A	182-B	#N/A	#N/A	Pump
PMP-8	203-A	203-B	#N/A	#N/A	Pump
PMP-9	204-A	204-B	#N/A	#N/A	Pump

TABLE 4.6: Energy Usage

Pump	Usage Factor	Avg. Effic.	Kw-hr /m3	Avg. Kw	Peak Kw	Cost /day
PMP-1	100.00	75.00	0.27	76.59	76.59	0.00
PMP-8	100.00	75.00	0.45	99.74	99.74	0.00
PMP-9	100.00	75.00	0.11	37.49	37.49	0.00

Table 4.7 shows the output results for the node. The demand shows the rate of withdrawal from the network, and the output result for nodes having negative demands indicates that water is entering into the network (Rossman, 2000). The result also shows that 5 of the nodes are having negative pressures which indicate that the network pressure is not enough to meet the total demand used.

TABLE 4.7: Node results

Node ID	Demand LPS	Head m	Pressure m	Quality
J-1	6.28	169.72	15.51	0.00

J-2	7.15	173.25	26.84	0.00	
J-3	13.58	161.45	11.84	0.00	
J-4	20.45	154.69	21.53	0.00	
J-5	19.00	159.39	18.61	0.00	
J-6	1.20	178.68	-8.93	0.00	
J-7	7.27	170.77	-0.23	0.00	
J-8	5.06	160.77	13.98	0.00	
J-9	32.41	151.17	17.60	0.00	
J-10	11.89	155.55	18.60	0.00	
J-11	13.91	160.42	15.49	0.00	
J-12	1.50	173.24	35.14	0.00	
J-13	37.25	166.72	53.93	0.00	
J-14	23.53	161.51	14.47	0.00	
J-15	11.89	172.93	-11.87	0.00	
J-16	10.07	161.84	12.06	0.00	
J-17	6.19	152.98	24.31	0.00	
J-18	10.65	164.04	27.67	0.00	
J-19	20.72	160.36	23.49	0.00	
J-20	2.25	162.57	15.88	0.00	
J-21	17.21	153.00	25.61	0.00	
J-22	11.27	152.27	-6.24	0.00	
J-23	1.13	171.59	8.75	0.00	
J-24	23.55	162.66	25.09	0.00	
J-25	8.11	166.68	31.62	0.00	
J-26	0.58	178.51	-8.86	0.00	
J-27	1.30	188.19	0.80	0.00	
182-A	0.00	139.26	-9.39	0.00	
182-B	0.00	213.23	64.42	0.00	
203-A	0.00	114.86	-52.31	0.00	
203-B	0.00	239.12	71.70	0.00	
204-A	0.00	148.06	-35.31	0.00	
204-B	0.00	179.36	-4.08	0.00	
R-1	-79.38	139.26	0.00	0.00	Reservoir
T-3	-50.21	191.55	8.38	0.00	Tank
T-5	-87.98	173.32	8.38	0.00	Tank
T-6	-107.82	171.42	8.38	0.00	Tank

TABLE 4.8: Output results for the link

Allowable Headloss m/km: 10, Velocity > 0.5m/s

Link	Flow	Velocity	Headloss	Status

ID	LPS	m/s	m/km	
P-1	-15.92	0.90	6.25	Open
P-2	57.57	1.83	16.66HI	Open
P-3	-31.69	1.01	5.51	Open
P-4	2.94	0.67	8.02	Open
P-5	-4.05	0.52	3.57	Open
P-6	2.85	0.65	7.59	Open
P-8	4.47	0.57	4.30	Open
P-9	8.43	0.48LO	1.93	Open
P-10	-27.04	1.53	16.69HI	Open
P-11	21.86	1.24	11.26HI	Open
P-12	-50.21	2.75	48.58	Open
P-13	27.33	0.56	1.41	Open
P-14	-5.67	0.08LO	0.03	Open
P-15	-50.24	0.71	1.80	Open
P-16	29.46	0.19	0.09	Open
P-17	6.39	0.04LO	0.01	Open
P-18	18.89	1.07	8.59	Open
P-19	-10.58	0.60	2.93	Open
P-20	4.89	1.11	20.59HI	Open
P-21	2.45	0.55	5.70	Open
P-22	137.91	1.95	11.65HI	Open
P-23	84.28	1.19	4.68	Open
P-24	52.51	1.07	4.74	Open
P-25	62.14	1.27	6.47	Open
P-26	11.26	0.16LO	0.11	Open
P-27	93.29	1.32	5.65	Open
P-28	-10.23	0.21LO	0.23	Open
P-29	-14.09	0.80	4.99	Open
P-30	-39.18	0.80	2.75	Open
P-31	85.45	1.21	4.80	Open
P-32	-17.66	1.00	7.58	Open
P-33	20.34	1.15	9.85	Open
P-34	-117.10	2.39	20.92HI	Open
P-35	2.68	0.15LO	0.23	Open
P-36	-46.22	0.94	3.74	Open
P-37	-54.22	1.10	5.03	Open
P-39	-25.84	0.37LO	0.52	Open
P-40	2.39	0.14LO	0.19	Open
P-41	17.44	0.99	7.40	Open
P-42	-58.27	1.19	5.74	Open
P-43	-43.67	0.89	3.37	Open
P-44	79.38	0.10LO	0.00	Open
P-45	79.38	4.35	113.50HI	Open
P-58	-61.54	3.48	76.52HI	Open
P-59	-61.54	3.48	76.52HI	Open
P-61	91.84	1.30	5.49	Open
P-62	91.84	1.30	5.49	Open
PMP-1	79.38	0.00	-73.97	Open Pump
PMP-8	61.54	0.00	-124.26	Open Pump
PMP-9	91.84	0.00	-31.30	Open Pump

In Table 4.8, the simulation output for the link shows that 9 of the pipes are having low velocity, which shows that the pipes has been oversized and should be reduced. The hydraulic implication of this low velocity is that stagnation of flow may occur, thereby causing the accumulation of debris in the affected pipes and this might lead to blockage flow. In addition, it results in pressure increase thereby causing the water column and the pipe walls in this section to somewhat compress; both of these phenomena help provide a little additional volume, allowing water to continuously enter the section until it comes to a complete stop. The increase in pressure due to low velocity in a pipe may fracture the pipe wall or cause other damage in the pipeline system at a later time. Matter will be sucked into a fractured pipe and this result in pollution of water supply.

The simulation output also depicts that 10 pipes are having high head loss. The headloss shows a limitation of flow to other parts of the network and it is caused by undersizing of pipes. The hydraulic implication of this high headloss is that frequent pipe burst is caused by higher velocity and headloss. The force resulting from high velocity and headloss may cause a pressure rise in the pipe greater than the normal static pressure of the pipe. This effect is known as water hammer and it causes pipe burst. In pipe sections where high headloss becomes unavoidable, thrust blocks should be provided at critical sections of the affected pipes to hold them in place in order to curtail the headloss

The flow also shows reverse direction in 20 of the pipes which indicate that the flow is in the anti clockwise direction to the assumed flow. This can be corrected by reversing the end nodes of the pipes and is always instrumental in re-orienting the links that were initially in the wrong direction. (Rossman, 2000).

Areas affected by this inadequacy are those along G.R.A, Ramatu Dangana axis and along Bida Town Hall on Ahmadu Bello way.

4.2 Modification of pipe diameters

In order to eliminate the problems of low velocity, high headloss and pressure, diameters of some pipes were modified. The pipes affected by the changes are shown in table 4.9. The new network was run in the Epanet program and the outcome is as shown in table 4.10 - 4.13

TABLE 4.9: Modified pipe diameters

Pipe No.	Original Diameter (mm)	New Diameter (mm)
2	200	300
9	150	200
10	150	300
11	150	300
12	150	300
14	300	250
17	450	350
20	75	300
22	300	350
26	300	200
28	250	200
34	250	300
35	150	100
39	300	250

40	150	100
44	1000	450
45	150	450
58	150	250
59	150	250

4.3 Hydraulic Simulation for the pipe Networks after modification

Table 4.10 shows the input results for the link after modification indicating the direction of flow for each of the node showing the start and end nodes and reflecting the modified diameter of each of the links and Table 4.11 shows the output of the energy consumption.

TABLE 4.10: Input results for the link after modification

Link ID	Start Node	End Node	Length m	Diameter mm
P-1	J-2	J-1	564	150
P-2	T-6	J-3	598	300
P-3	J-5	J-4	851	200
P-4	J-8	J-9	1197	75
P-5	J-11	J-10	1365	100
P-6	J-13	J-4	1584	75
P-8	J-5	J-13	1706	100
P-9	J-18	J-19	1906	200
P-10	J-6	J-27	570	300
P-11	J-27	J-7	1548	300
P-12	J-27	T-3	69	150
P-13	J-19	J-24	1629	250
P-14	J-13	J-25	1336	250
P-15	T-6	J-13	2618	300
P-16	T-5	J-2	802	450
P-17	J-2	J-12	682	350
P-18	J-16	J-9	1243	150

P-19	J-9	J-17	620	150
P-20	J-12	J-25	319	300
P-21	J-21	J-25	2400	75
P-22	T-5	J-23	149	350
P-23	J-23	J-20	1926	300
P-24	J-23	J-1	394	250
P-25	J-1	J-24	1090	250
P-26	J-24	J-20	840	200
P-27	J-20	J-10	1243	300
P-28	J-3	J-16	1677	200
P-29	J-4	J-17	343	150
P-30	J-18	J-16	798	250
P-31	J-10	J-22	682	300
P-32	J-14	J-22	1218	150
P-33	J-14	J-11	110	150
P-34	J-26	J-15	266	300
P-35	J-17	J-21	72	100
P-36	J-8	J-5	370	250
P-37	J-3	J-8	136	250
P-38	J-22	J-26	921	300
P-39	J-26	J-6	335	250
P-40	J-11	J-19	336	100
P-41	J-19	J-21	994	150
P-42	J-7	J-18	1172	250
P-43	J-15	J-7	644	250
P-44	R-1	182-A	0.001	450
P-45	182-B	T-5	352	450
P-58	203-B	J-14	1014	250
P-59	J-15	203-A	759	250
P-61	J-22	204-A	767	300
P-62	204-B	J-26	155	300
PMP-1	182-A	182-B	#N/A	#N/A Pump
PMP-8	203-A	203-B	#N/A	#N/A Pump
PMP-9	204-A	204-B	#N/A	#N/A Pump

TABLE 4.11: Energy Usage

Pump	Usage Factor	Avg. Effic.	Kw-hr /m3	Avg. Kw	Peak Kw	Cost /day
PMP-1	100.00	75.00	0.12	40.75	40.75	0.00
PMP-8	100.00	75.00	0.09	30.58	30.58	0.00
PMP-9	100.00	75.00	0.12	40.12	40.12	0.00

Table 4.12 shows the new output results for the node. The results shows that the negative pressures have been eliminated and the new network is sufficient to meet the demand since most of the

pressures are not in excess of 10 m of water and are not above the allowable pressure for the pipe material used.

TABLE 4.12: Node results

Node ID	Demand LPS	Head m	Pressure m	Quality
J-1	6.28	171.97	17.76	0.00
J-2	7.15	173.10	26.69	0.00
J-3	13.58	170.14	20.51	0.00
J-4	20.45	163.29	30.10	0.00
J-5	19.00	167.94	27.15	0.00
J-6	1.20	190.52	22.84	0.00
J-7	7.27	186.60	35.53	0.00
J-8	5.06	169.42	22.61	0.00

J-9	32.41	160.27	26.69	0.00	
J-10	11.89	163.34	26.38	0.00	
J-11	13.91	184.19	39.20	0.00	
J-12	1.50	172.77	34.66	0.00	
J-13	37.25	170.81	58.01	0.00	
J-14	23.53	187.74	40.65	0.00	
J-15	11.89	187.14	28.26	0.00	
J-16	10.07	172.24	22.44	0.00	
J-17	6.19	161.74	33.05	0.00	
J-18	10.65	175.01	38.62	0.00	
J-19	20.72	169.14	32.25	0.00	
J-20	2.25	167.93	21.23	0.00	
J-21	17.21	161.83	34.43	0.00	
J-22	11.27	161.01	32.42	0.00	
J-23	1.13	172.89	17.03	0.00	
J-24	23.55	169.14	31.55	0.00	
J-25	8.11	172.46	37.40	0.00	
J-26	0.58	189.76	21.33	0.00	
J-27	1.30	191.08	33.63	0.00	
182-A	0.00	139.26	-9.39	0.00	
182-B	0.00	173.58	24.86	0.00	
203-A	0.00	176.67	9.37	0.00	
203-B	0.00	201.73	34.39	0.00	
204-A	0.00	156.86	-26.53	0.00	
204-B	0.00	190.60	7.14	0.00	
R-1	-91.02	139.26	0.00	0.00	Reservoir
T-3	-102.76	191.55	8.38	0.00	Tank
T-5	-59.75	173.32	8.38	0.00	Tank
T-6	-71.87	171.42	8.38	0.00	Tank

Table 4.13 shows the new output results for links after modification. The results did not yield a better result in changing the number of pipes having low velocity and high headloss. The remedy to this problem is to install washouts at the affected locations which will facilitate the emptying of the main and removal of stagnant or dirty water.

TABLE 4.13: Output results for the link

Allowable Headloss m/km: 10, Velocity > 0.5m/s

Link ID	Flow LPS	Velocity m/s	Headloss m/km	Status

P-1	8.59	0.49LO	2.00	Open
P-2	55.17	0.78	2.14	Open
P-3	31.57	1.00	5.48	Open
P-4	2.87	0.65	7.65	Open
P-5	8.87	1.13	15.27HI	Open
P-6	2.22	0.50	4.75	Open
P-8	2.69	0.34LO	1.68	Open
P-9	23.13	0.74	3.08	Open
P-10	36.47	0.52	0.99	Open
P-11	64.99	0.92	2.89	Open
P-12	102.76	1.45	6.76	Open
P-13	0.91	0.02LO	0.00	Open
P-14	25.46	0.52	1.24	Open
P-15	16.70	0.24LO	0.23	Open
P-16	52.95	0.33LO	0.27	Open
P-17	37.21	0.39LO	0.49	Open
P-18	20.10	1.14	9.63	Open
P-19	9.44	0.53	2.38	Open
P-20	35.71	0.51	0.95	Open
P-21	2.13	0.48LO	4.43	Open
P-22	97.82	1.02	2.91	Open
P-23	61.02	0.86	2.57	Open
P-24	35.68	0.73	2.32	Open
P-25	37.99	0.77	2.60	Open
P-26	15.34	0.49LO	1.44	Open
P-27	74.11	1.05	3.69	Open
P-28	14.22	0.45LO	1.25	Open
P-29	13.34	0.75	4.51	Open
P-30	44.38	0.90	3.47	Open
P-31	71.10	1.01	3.42	Open
P-32	31.35	1.77	21.94HI	Open
P-33	38.64	2.19	32.32HI	Open
P-34	125.86	1.78	9.84	Open
P-35	2.30	0.29LO	1.25	Open
P-36	47.88	0.98	3.99	Open
P-37	55.81	1.14	5.30	Open
P-39	35.27	0.72	2.27	Open
P-40	15.86	2.02	44.78HI	Open
P-41	17.37	0.98	7.35	Open
P-42	78.17	1.59	9.90	Open
P-43	20.45	0.42LO	0.83	Open
P-44	91.02	0.57	0.00	Open
P-45	91.02	0.57	0.75	Open
P-58	93.52	1.91	13.80HI	Open
P-59	93.52	1.91	13.80HI	Open
P-61	91.18	1.29	5.42	Open
P-62	91.18	1.29	5.42	Open
PMP-1	91.02	0.00	-34.32	Open Pump
PMP-8	93.52	0.00	-25.07	Open Pump
PMP-9	91.18	0.00	-33.74	Open Pump

4.4 Future Scenario

Future design period of 30 years was considered. The population distribution of the area in the sectors with reference to the proposed network loops for the year 2036 was calculated and shown on Table 4.14

TABLE 4.14: Area and Population Distribution in the Loop for year 2036

Loop No.	**Area of the Loop (hecter)**	**Population per Loop**	**Density (Population/Area)**	**Water demand for the loop (m^3/s)**
1.	16.4	3,706	225.98	0.014
2.	80.7	7,413	91.86	0.030
3.	327.8	51,894	158.31	0.210
4.	169.3	44,481	262.73	0.180
5.	55.2	18,534	335.76	0.074
6.	100.2	29,653	295.94	0.120
7.	49.8	11,121	223.31	0.044
8.	125	40,775	326.20	0.164
9.	174.6	48,187	275.99	0.194
10.	63.6	22,241	349.70	0.090
11.	138.7	37,068	267.25	0.150
12.	82.1	33,362	406.36	0.134

13.	48.4	14,828	306.36	0.060
14.	83.8	25,947	309.63	0.104
TOTAL	**1,515.60**	**389,209**	**3,835.38**	**1.568**

Demands were assigned to individual nodes and the network was analyzed with all its parameters and tested, the result is as shown in Tables 4.15 – 4.18

4.5 Hydraulic Simulation for the pipe Networks for future population demand

Table 4.15 shows the input results for the link based on future scenario indicating the direction of flow for each of the node, the length and the diameter of each of the links while Table 4.16 shows the output of the energy consumption of the pumps.

TABLE 4.15: Input results for the link for year 2036

Link ID	Start Node	End Node	Length m	Diameter mm
P-1	J-2	J-1	564	150
P-2	T-6	J-3	598	300
P-3	J-5	J-4	851	200
P-4	J-8	J-9	1197	75
P-5	J-11	J-10	1365	100
P-6	J-13	J-4	1584	75
P-8	J-5	J-13	1706	100
P-9	J-18	J-19	1906	200
P-10	J-6	J-27	570	300
P-11	J-27	J-7	1548	300
P-12	J-27	T-3	69	150
P-13	J-19	J-24	1629	250
P-14	J-13	J-25	1336	250
P-15	T-6	J-13	2618	300
P-16	T-5	J-2	802	450
P-17	J-2	J-12	682	350
P-18	J-16	J-9	1243	150
P-19	J-9	J-17	620	150
P-20	J-12	J-25	319	300
P-21	J-21	J-25	2400	75
P-22	T-5	J-23	149	350
P-23	J-23	J-20	1926	300
P-24	J-23	J-1	394	250
P-25	J-1	J-24	1090	250
P-26	J-24	J-20	840	200
P-27	J-20	J-10	1243	300
P-28	J-3	J-16	1677	200

P-29	J-4	J-17	343	150	
P-30	J-18	J-16	798	250	
P-31	J-10	J-22	682	300	
P-32	J-14	J-22	1218	150	
P-33	J-14	J-11	110	150	
P-34	J-26	J-15	266	300	
P-35	J-17	J-21	72	100	
P-36	J-8	J-5	370	250	
P-37	J-3	J-8	136	250	
P-38	J-22	J-26	921	300	
P-39	J-26	J-6	335	250	
P-40	J-11	J-19	336	100	
P-41	J-19	J-21	994	150	
P-42	J-7	J-18	1172	250	
P-43	J-15	J-7	644	250	
P-44	R-1	182-A	0.001	450	
P-45	182-B	T-5	352	450	
P-58	203-B	J-14	1014	250	
P-59	J-15	203-A	759	250	
P-61	J-22	204-A	767	300	
P-62	204-B	J-26	155	300	
PMP-1	182-A	182-B	#N/A	#N/A	Pump
PMP-8	203-A	203-B	#N/A	#N/A	Pump
PMP-9	204-A	204-B	#N/A	#N/A	Pump

TABLE 4.16: Energy Usage

Pump	Usage Factor	Avg. Effic.	Kw-hr /m3	Avg. Kw	Peak Kw	Cost /day
PMP-1	100.00	75.00	0.12	40.75	40.75	0.00
PMP-8	100.00	75.00	0.09	30.58	30.58	0.00
PMP-9	100.00	75.00	0.12	40.12	40.12	0.00

The output results for the node for year 2036 are shown in Table 4.17. There is no significant difference from the output result in Table 4.12. Negative pressures are not experienced and the network will be sufficient to meet the demand.

TABLE 4.17: Node results for year 2036

Node ID	Demand LPS	Head m	Pressure m	Quality
J-1	6.28	171.97	17.76	0.00
J-2	7.15	173.10	26.69	0.00
J-3	13.58	170.14	20.51	0.00
J-4	20.45	163.29	30.10	0.00
J-5	19.00	167.94	27.15	0.00
J-6	1.20	190.52	22.84	0.00
J-7	7.27	186.60	35.53	0.00
J-8	5.06	169.42	22.61	0.00
J-9	32.41	160.27	26.69	0.00
J-10	11.89	163.34	26.38	0.00
J-11	13.91	184.19	39.20	0.00
J-12	1.50	172.77	34.66	0.00
J-13	37.25	170.81	58.01	0.00
J-14	23.53	187.74	40.65	0.00
J-15	11.89	187.14	28.26	0.00
J-16	10.07	172.24	22.44	0.00
J-17	6.19	161.74	33.05	0.00
J-18	10.65	175.01	38.62	0.00
J-19	20.72	169.14	32.25	0.00

J-20	2.25	167.93	21.23	0.00	
J-21	17.21	161.83	34.43	0.00	
J-22	11.27	161.01	32.42	0.00	
J-23	1.13	172.89	17.03	0.00	
J-24	23.55	169.14	31.55	0.00	
J-25	8.11	172.46	37.40	0.00	
J-26	0.58	189.76	21.33	0.00	
J-27	1.30	191.08	33.63	0.00	
182-A	0.00	139.26	-9.39	0.00	
182-B	0.00	173.58	24.86	0.00	
203-A	0.00	176.67	9.37	0.00	
203-B	0.00	201.73	34.39	0.00	
204-A	0.00	156.86	-26.53	0.00	
204-B	0.00	190.60	7.14	0.00	
R-1	-91.02	139.26	0.00	0.00	Reservoir
T-3	-102.76	191.55	8.38	0.00	Tank
T-5	-59.75	173.32	8.38	0.00	Tank
T-6	-71.87	171.42	8.38	0.00	Tank

Table 4.18 shows the future scenario for output results for links having no significant difference from that displayed in Table 4.13. The installed washouts proposed at the affected locations shall help in emptying the main and in removing stagnant or dirty water. The output results indicate that the network is also sufficient to meet the demand of the future population.

TABLE 4.18: Output results for the link for year 2036

Link ID	Flow LPS	Velocity m/s	Headloss m/km	Status
P-1	8.59	0.49LO	2.00	Open
P-2	55.17	0.78	2.14	Open
P-3	31.57	1.00	5.48	Open
P-4	2.87	0.65	7.65	Open
P-5	8.87	1.13	15.27HI	Open
P-6	2.22	0.50	4.75	Open
P-8	2.69	0.34LO	1.68	Open
P-9	23.13	0.74	3.08	Open
P-10	36.47	0.52	0.99	Open
P-11	64.99	0.92	2.89	Open
P-12	102.76	1.45	6.76	Open
P-13	0.91	0.02LO	0.00	Open
P-14	25.46	0.52	1.24	Open
P-15	16.70	0.24LO	0.23	Open
P-16	52.95	0.33LO	0.27	Open

P-17	37.21	0.39LO	0.49	Open
P-18	20.10	1.14	9.63	Open
P-19	9.44	0.53	2.38	Open
P-20	35.71	0.51	0.95	Open
P-21	2.13	0.48LO	4.43	Open
P-22	97.82	1.02	2.91	Open
P-23	61.02	0.86	2.57	Open
P-24	35.68	0.73	2.32	Open
P-25	37.99	0.77	2.60	Open
P-26	15.34	0.49	1.44	Open
P-27	74.11	1.05	3.69	Open
P-28	14.22	0.45LO	1.25	Open
P-29	13.34	0.75	4.51	Open
P-30	44.38	0.90	3.47	Open
P-31	71.10	1.01	3.42	Open
P-32	31.35	1.77	21.94HI	Open
P-33	38.64	2.19	32.32HI	Open
P-34	125.86	1.78	9.84	Open
P-35	2.30	0.29LO	1.25	Open
P-36	47.88	0.98	3.99	Open
P-37	55.81	1.14	5.30	Open
P-39	35.27	0.72	2.27	Open
P-40	15.86	2.02	44.78HI	Open
P-41	17.37	0.98	7.35	Open
P-42	78.17	1.59	9.90	Open
P-43	20.45	0.42LO	0.83	Open
P-44	91.02	0.57	0.00	Open
P-45	91.02	0.57	0.75	Open
P-58	93.52	1.91	13.80HI	Open
P-59	93.52	1.91	13.80HI	Open
P-61	91.18	1.29	5.42	Open
P-62	91.18	1.29	5.42	Open
PMP-1	91.02	0.00	-34.32	Open Pump
PMP-8	93.52	0.00	-25.07	Open Pump
PMP-9	91.18	0.00	-33.74	Open Pump

4.6 Comparison between the present and future headloss and velocity

The headloss and velocity in the present and future situation were compared in order to know the improvement or otherwise of the network and plan for necessary future adjustment. Table 4.19 shows the differences in the two situations.

Table 4.19: Present and Future Headloss and Velocity

Link ID	Present Headloss	Future Headloss	Present Velocity	Future Velocity
P-1	6.25	2.00	0.90	0.49LO

P-2	16.66HI	2.14	1.83	0.78
P-3	5.51	5.48	1.01	1.00
P-4	8.02	7.65	0.67	0.65
P-5	3.57	15.27HI	0.52	1.13
P-6	7.59	4.75	0.65	0.50
P-8	4.30	1.68	0.57	0.34LO
P-9	1.93	3.08	0.48LO	0.74
P-10	16.69HI	0.99	1.53	0.52
P-11	11.26	2.89	1.24	0.92
P-12	48.58HI	6.76	2.75	1.45
P-13	1.41	0.00	0.56	0.02LO
P-14	0.03	1.24	0.08LO	0.52
P-15	1.80	0.23	0.71	0.24LO
P-16	0.09	0.27	0.19LO	0.33LO
P-17	0.01	0.49	0.04LO	0.39LO
P-18	8.59	9.63	1.07	1.14
P-19	2.93	2.38	0.60	0.53
P-20	20.59HI	0.95	1.11	0.51
P-21	5.70	4.43	0.55	0.48LO

P-22	11.65HI	2.91	1.95	1.02
P-23	4.68	2.57	1.19	0.86
P-24	4.74	2.32	1.07	0.73
P-25	6.47	2.60	1.27	0.77
P-26	0.11	1.44	0.16LO	0.49LO
P-27	5.65	3.69	1.32	1.05

Table 4.19: Present and Future Headloss and Velocity

Link ID	Present Headloss	Future Headloss	Present Velocity	Future Velocity
P-28	0.23	1.25	0.21LO	0.45LO
P-29	4.99	4.51	0.80	0.75
P-30	2.75	3.47	0.80	0.90
P-31	4.80	3.42	1.21	1.01
P-32	7.58	21.94HI	1.00	1.77
P-33	9.85	32.32HI	1.15	2.19
P-34	20.92HI	9.84	2.39	1.78
P-35	0.23	1.25	0.15LO	0.29LO
P-36	3.74	3.99	0.94	0.98
P-37	5.03	5.30	1.10	1.14
P-39	0.52	2.27	0.37LO	0.72
P-40	0.19	44.78HI	0.14LO	2.02
P-41	7.40	7.35	0.99	0.98
P-42	5.74	9.90	1.19	1.59
P-43	3.37	0.83	0.89	0.42

P-44	0.00	0.00	0.10LO	0.57
P-45	113.50LO	0.75	4.35	0.57
P-58	76.52HI	13.80	3.48	1.91
P-60	76.52HI	13.80	3.48	1.91
P-61	5.49	5.42	1.30	1.29
P-62	5.49	5.42	1.30	1.29
TOTAL	**9**	**6**	**10**	**10**

NOTE: HI is High Headloss and LO is Low Velocity

Table 4.19 is represented in figures 4.2 and 4.3 showing the relationship between head losses and velocities in the existing and future scenario. The relationship in the charts shows considerable reduction in the number of pipes with high headloss from 9 in the existing situation to 6 in future situation. The numbers of pipes with low velocity, however, remains the same for both present and future situation.

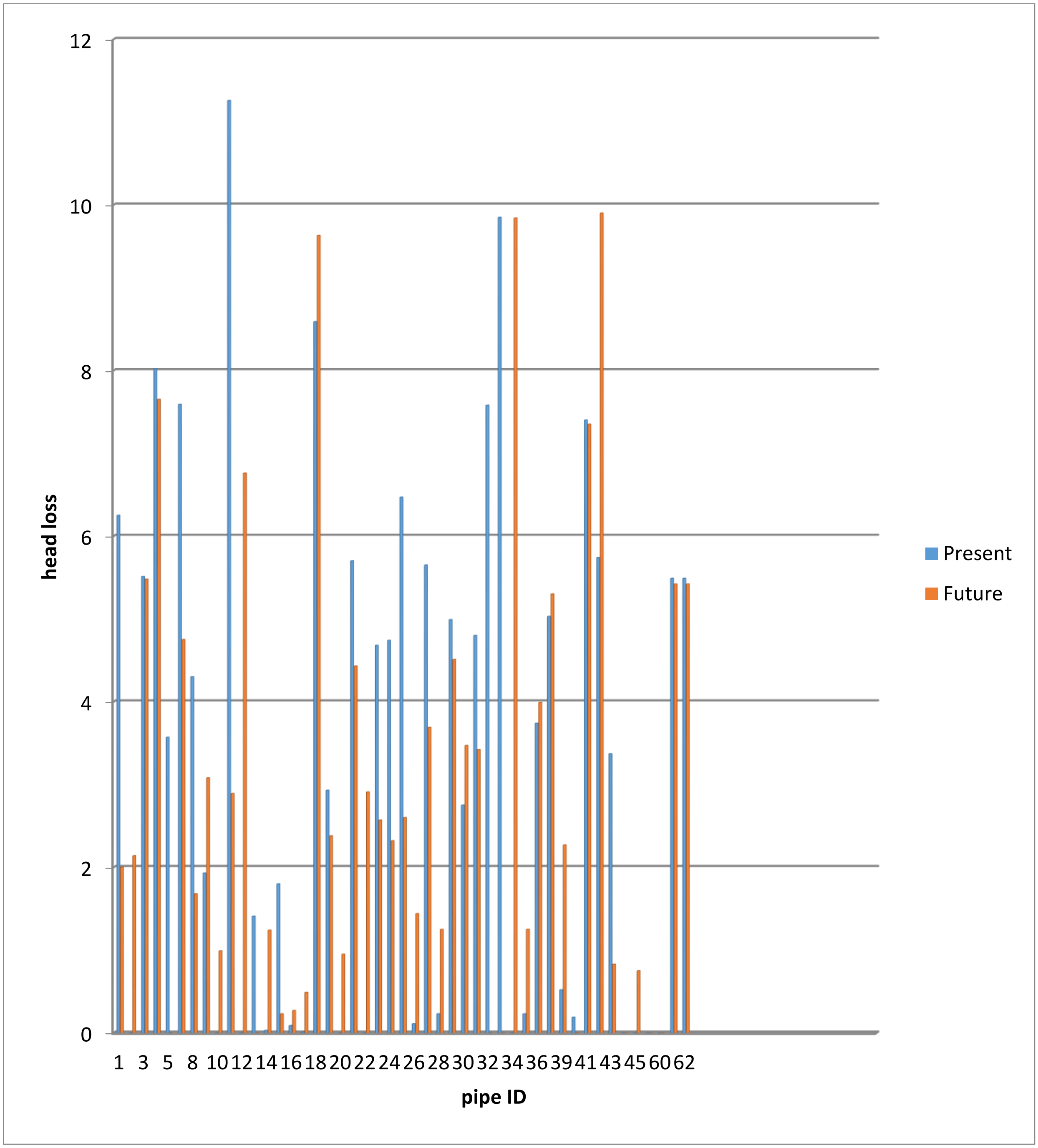
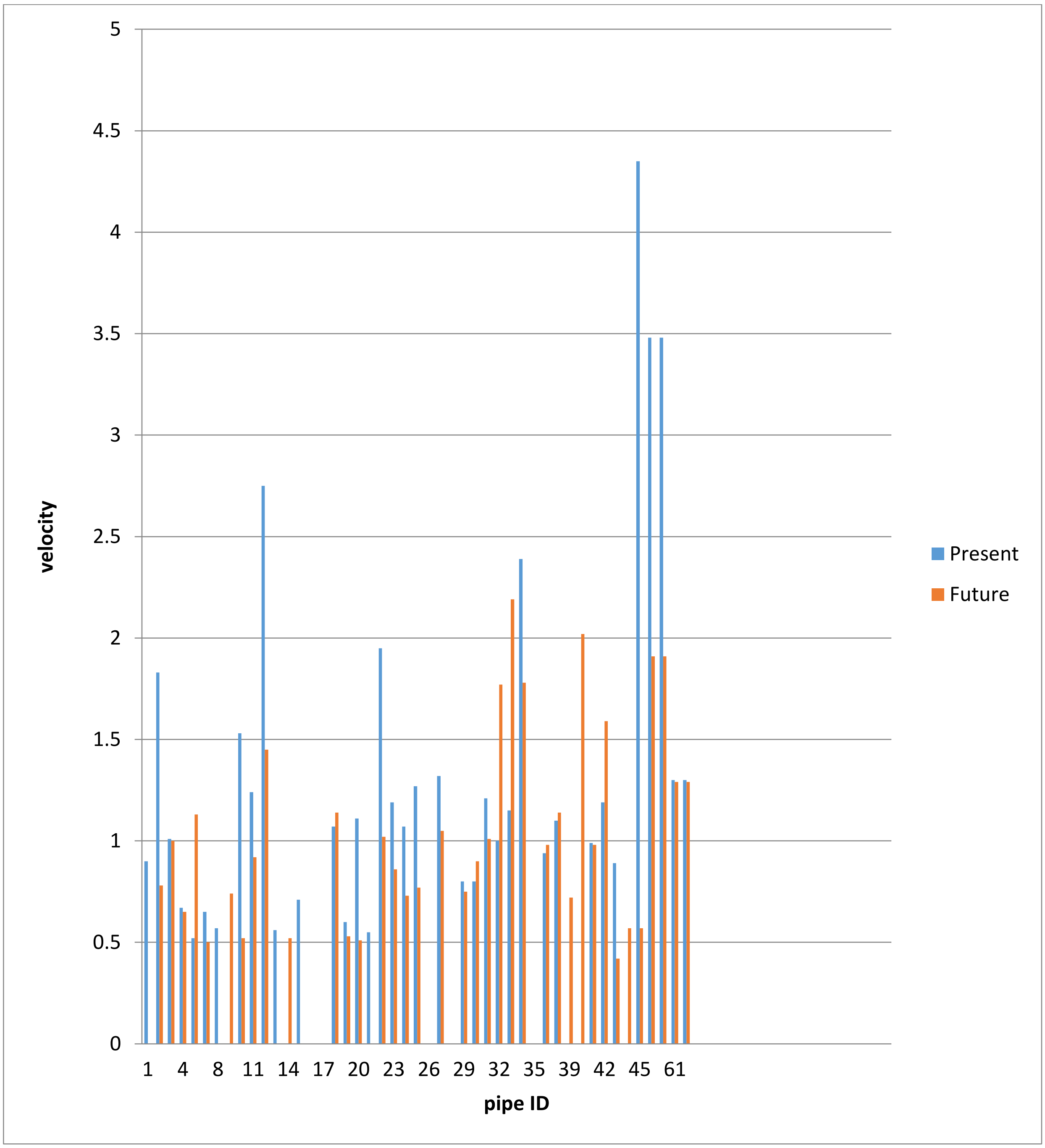


Figure 4.2 **Head losses for existing and future scenario**



5
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
velocity
1
4
8
11
14
17
20
23
26
29
32
35
39
42
45
61
pipe ID
Present
Future

Figure 4.3 Velocity for existing and future scenario

CHAPTER FIVE

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusion

The present situation of Bida distribution system is inefficient in meeting with the demand of the consumers. The design capacity is not adequate for the present population and the situation is compounded by inadequate supply of water from the production point to the system

The distribution network need to be modified to cope with the present and the future water demand of the inhabitants of Bida Town even if there is adequate supply from the production unit.

5.2 Recommendations

a. In order to meet up with the future demand and population challenges there is need for changes in some pipe diameters as solution to low velocity and high headloss experienced.

b. In pipe sections where high headloss becomes unavoidable, thrust blocks should be provided at critical sections of the affected pipes.

c. Washouts help in emptying and removing stagnant or dirty water from the mains especially where low velocity and high Headloss are eminent.

d. Urban development should be managed such that population of some loops is not over stretched.

5.3 Areas of future research

Based on the limitation experienced in the use of this software, there is need to conduct intermittent simulation for the specific hour in future study. This can be achieved by creating a time pattern that will made demand at the nodes vary in a periodic way over the cause of a day.

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