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DESIGN OF A WATER SUPPLY DISTRIBUTION NETWORK USING EPANET 2.0: A CASE STUDY OF MAIDUGURI ZONE 3, NIGERIA

A. N. Alkali, S. G. Yadima, B. Usman, U. A. Ibrahim and A. G. Lawan
(Department of Civil and Water Resources Engineering, University of Maiduguri, Nigeria)
*Corresponding author's email: abdulhamidalkali@yahoo.com

Abstract
Maiduguri Water Supply Scheme designed by its consultants in 1983 has divided the town into five water supply distribution zones. Zone 3 is one of three remaining zones in the city without a distribution network supplied from the Maiduguri water treatment plant. Consequently, residents obtain water from local water vendors, who retrieve the water from shallow boreholes which has proven to be tedious, cost ineffective and physically exhausting. A water supply network was proposed with the aid of EPANET software and the results obtained conformed to the design criteria established by the previous designers of the Scheme. A total water demand, projected for the year 2031 of 420LPS, minimum pressure head of 14.08m and maximum velocity of flow in the pipes of 1.24m/s was obtained. The use of EPANET software has proven to be time saving and its application is especially for the analysis of larger distribution networks is recommended.

Keywords: Water, distribution, Maiduguri, Zone.

1. Introduction

Water is undoubtedly one of the most essential commodities every living creature requires for survival. Its purpose in commercial, farming or irrigation, navigation, recreation and of course fundamentally for domestic purposes makes it a valuable commodity with no other alternative. Water is unequally distributed on the earth and its availability at any place varies greatly with time and size. The total supplies of fresh water on earth far exceed human demand. About 2.8% of the total amount of water on earth is fresh water of which 0.6% is groundwater. The rest is available in the seas and oceans while a negligible amount as soil moisture. Out of the 2.2%, 2.15% is found in the form of ice sheets and glaciers while the remaining 0.05% is available as surface runoff (Mustafa and Yusuf, 2012). One of the major challenges hampering the establishment of an efficient water supply system is the incapacity to provide sufficient amount of water to meet the requirement of the ever growing population. A rapid increase in population and technological advancement all over the world compels a corresponding rise in the demand for potable water. As such, water policy makers rely on population projections to assess future demands for this much needed commodity. These projections are necessitated by the population change with time as a result of births and the arrival of migrants which add individuals to a population, whereas deaths and the departure of migrants subtract individuals from a population (Kaneda and Bremner, 2014). Notable of these projection techniques include the Arithmetic or Linear, Geometric and Graphical methods (Alcantara, 2002).

The projected population is a pertinent tool which if used assiduously would yield an optimum water demand for any given settlement (Nauges and Whittington, 2006). Several researchers and institutions have developed values for water consumption per capita (per head) based on the nature and quantity of usage viz; domestic, commercial, institutional or industrial settlements (Sanderson, 1998). For a municipal water supply system, the designer may be compelled to key- in at least three of the aforementioned. In addition, an efficient system must not overlook water for special services such as street cleaning, selling of water to contractors for erecting buildings, parks and recreation and fire-fighting. In some extreme cases, adjacent neighborhoods are not privileged to have water in their community. The per capita water demand would range in the settlement under study and thus necessitate an average value that would compensate shortfalls in cosmopolitan and commercial areas which are believed to consume a larger portion of water in the settlement (Piper, 2000).In a municipal settlement with high rise buildings, businesses and small irrigation activities, access to clean water is necessary and requires high capital investment in storage, treatment and conveyance networks in order to pave way for a smooth coordination of the activities of the society. The source of water for some parts of Maiduguri and Jere is from shallow boreholes conveyed by local water vendors which do not reach the consumers at the time and place most desired. Sufficient quantity needed for certain purposes may attract more costs as the vendors have to move to and fro to satisfy the needs of their consumers. Lack of available pipe borne water poses a great sanitary risk of sanitation and drinking water. Sufficient amount of water is required to flush away lingering pathogens in hospitals and public places, if the spread of diseases is to be prevented.

A water system has two primary requirements. Firstly, it needs to deliver adequate amounts of water to meet consumption requirements plus needed free flow requirements. Secondly, the water system needs to be reliable; the required amount of water needs to be available 24 hours a day (Tahal, 1995).Significant progress has been made in recent times in various aspects of water supply and distribution. Remote sensing techniques have been used in determining leakages in pipes (Agapiou, 2013). In a similar vein, the use of GIS has been used to study movement of surface and underground water resources (Cheyapalan *et. al.*, 2014; Harding, 2007). Breakthroughs have been made by mapping storm water using modern techniques such as Penetrating radar (Bell, 2014). On the conventional aspect, Excel- solver, which is known for its diversity in solving a wide range of complex problems, is a valuable tool in the analysis of water distribution (Banerjee *et. al.*, 2013). Modeling tools have been used to simplify numerous real life situations some of which are the Shuffled Complex Evolution to determine an optimum design of a distribution network (El-Sheikh *et. al.*, 2013; Atiquzzaman and Liong, 2004). The advent of EPANET (Environmental Protection Agency Network) has yielded substantive results in the determination of sufficient water requirement for an area (Arunkumar and Marriappan, 2011). The software was developed by Rossman (1994) for the United States Environmental Protection Agency (Lawan, 2016). EPANET is also efficient in determining the optimum velocities and discharges in pipes and pressures at junctions (Abubakar 2013; Newbold, 2009).Therefore, water supply networks through piping though expensive and tedious, ensure a safe delivery of water with little or no contamination. Thus, an efficient water distribution network should contain pipe systems, pumping stations, storage facilities, fire hydrants, house service connections, meters, booster pumps and other appurtenances (Abubakar *et. al.*, 2013).

2. Methodology

2.1 The Study Area

Maiduguri is the state capital of Borno State, North Eastern Nigeria. The state lies within latitude and longitude of the state is 11°51'N and 13°40'E respectively with an Altitude of 300m above mean sea level and borders with Chad, Cameroun and Niger (Olofin, 1997). The land has an area of 543km² and has a population of about 357,104 people. It is within the semi-arid climatic zone referred to as the SAHEL zone.The city practically experiences two distinct climatic seasons yearly. These are; a short rainy season usually from the month of June to September and a long dry season from October to May. The hottest months in the year are March, April and May having temperatures ranging between 30°C-43°C.

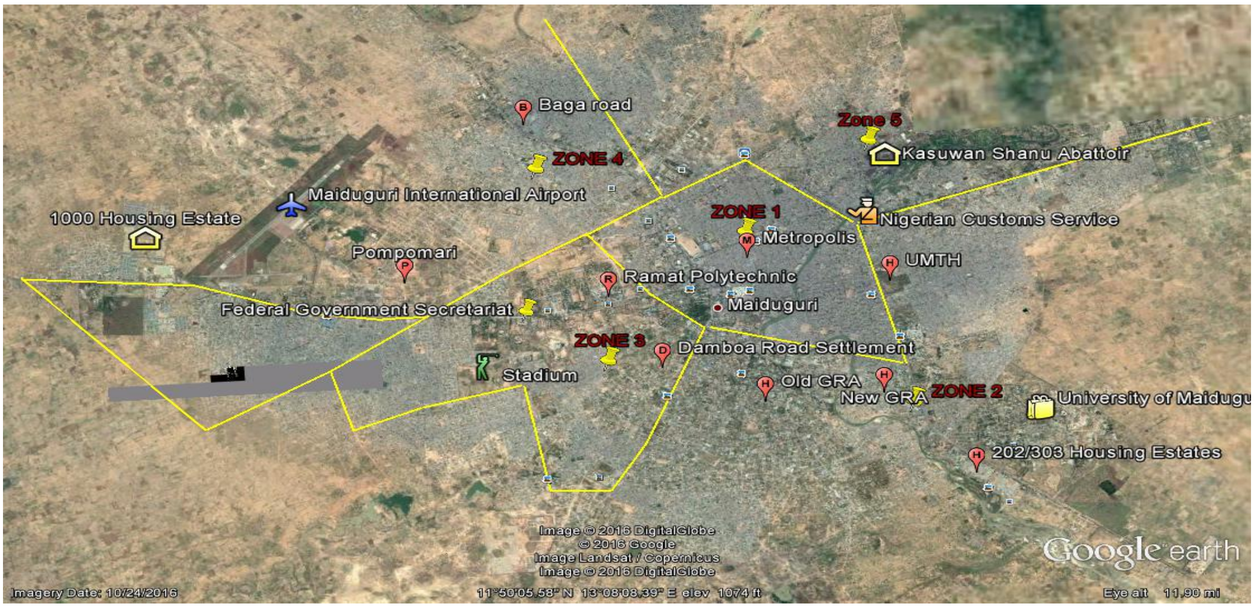

Figure 1: Map of Maiduguri, Borno State

Maiduguri and Jere have been divided into five distribution zones by the designers of the Maiduguri Water Supply Scheme (Tahal, 1995).The area of study, Zone 3 wards are located along Kano road in Maiduguri urban area.The zone lies in the north- west part of Maiduguri. The main areas in the zone are Bulumkutu, Gomari, Coca- cola industrial layout, the new stadium and all tertiary institutions in Kano road. Zone 3 is one of the three remaining zones without a water supply distribution network from Maiduguri Water Treatment Plant. This can be seen in Figure 2;

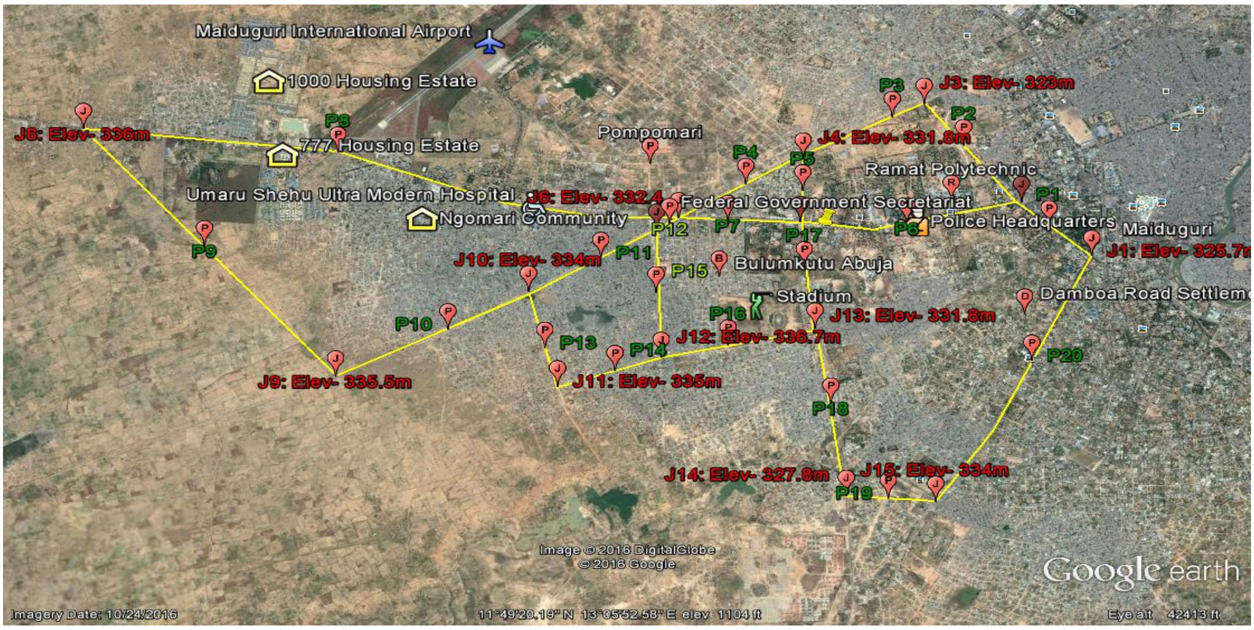

Figure 2: Map of Maiduguri – Zone 3

The others are zones 4 and 5 largely bounded by Baga road up to the Nigerian Customs office extending towards Chad Basin Development Authority office. The most populated zones are Zone 1 consisting of the metropolitan area and Zone 2 which consists of areas that include the University of Maiduguri from the East up to Damboa road in the West; Dalori quarters in the south up to the Customs office in the North.

2.2 Application of EPANET for Zone 3 Water distribution

The population of 103,500 for Zone 3 was obtained from the exercise conducted by the consultants in 1995 and was projected using the geometric growth rate formula i.ePt = Pi(1+r)n; where Pi = initial population, r = annual growth rate, n = number of years and Pt = Projected population (UNESCO, 2008). An annual growth rate of 3.5% (Tahal Consultants, 1995) was used and multiplied by 100 litres per Capita per Day (LCPD) for a design life of 2031 in line with the Sustainable Development Goals (Abdullahi*et al*., 2014). The map of the study area was obtained from Google Earth (See Figure 2) and the EPANET analysis was then carried out using the procedure mentioned thus;

1. Edit the properties such as the length of pipes, elevation, demand, reservoirs etc.

2. Select a set of analysis options

3. Run a hydraulic analysis. See figure 1;

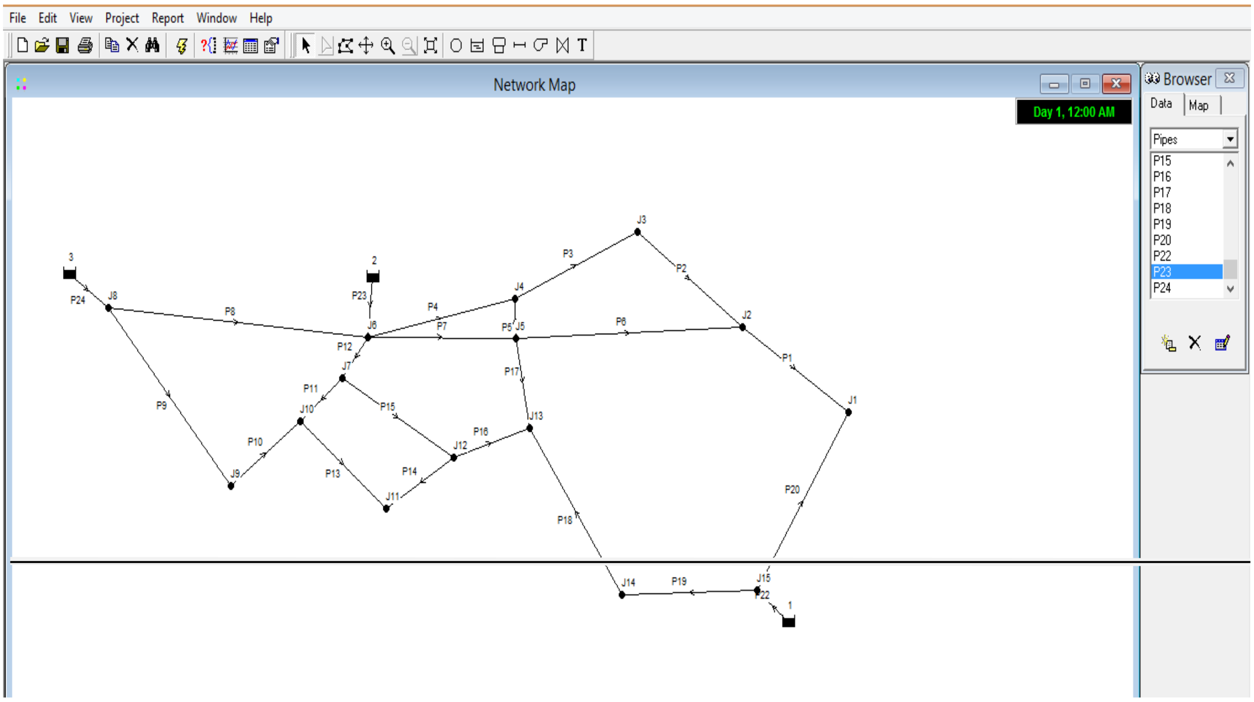

Figure 3: EPANET Network Map

1. Results and Discussion

The projected population for the year 2031 was found to be 357,075 and thus a water demand for zone 3 as 35,707,500litres per day. This value is approximately equivalent to 420 litres per second (LPS). The pipes conveying water into the system have been designed to meet the criteria stipulated by Tahal Consultants (1995) and presented in Table 1. The diameters of 300mm were specified by the design criteria for primary pipes (Tahal, 1995). All Primary distribution pipes in the system have conformed to this criterion except for pipes P22, P23 and P24 which have all been increased to 400mm. According to Featherstone and Nalluri (1995) from the continuity equation, i.e $A_1V_1= A_2V_2$, where A= cross- sectional area of pipe and V= Velocity of flow, an increase in pipe diameter would correspond to a decrease in velocity of flow within the pipe. These increments in cross sectional area of the pipes were necessary since the pipes supply water from the reservoirs into the distribution network.

Table 1: Flow and velocity of network links

Link ID	Length (m)	Diameter (mm)	Flow (Ltrs/Sec)	Velocity (m/s)	Unit Head Loss(m/km)
Pipe P1	1102.34	300	-11.83	0.17	0.10
Pipe P2	1734.48	300	-12.46	0.18	0.11
Pipe P3	1617.16	300	-22.46	0.32	0.31
Pipe P4	1705.64	300	-43.84	0.62	1.03
Pipe P5	919.32	300	11.39	0.16	0.09
Pipe P6	2537.00	300	19.38	0.27	0.24
Pipe P7	1430.66	300	-49.61	0.70	1.29
Pipe P8	7143.00	300	-25.90	0.37	0.40
Pipe P9	4480.00	300	48.43	0.69	1.24
PipeP10	2569.00	300	-1.57	0.02	0.01
PipeP11	2012.29	300	-37.27	0.53	0.77
PipeP12	308.00	300	-87.93	1.24	3.64
PipeP13	1319.00	300	0.70	0.01	0.01
PipeP14	1272.44	300	-19.30	0.27	0.24
PipeP15	1755.23	300	-35.67	0.50	0.71
PipeP16	1823.00	300	-8.64	0.12	0.06
PipeP17	1451.56	300	-21.61	0.31	0.29
PipeP18	2272.65	300	-12.02	0.17	0.10
PipeP19	1047.00	350	-47.02	0.67	1.17
PipeP20	3762.34	375	28.17	0.40	0.47
PipeP22	50.00	400	170.48	1.36	3.04
PipeP23	50.00	400	129.33	1.03	1.84
PipeP24	50.00	400	120.19	0.96	1.61

The pipes possessing higher magnitudes of flows reflect the proximity of these pipes to the source of supply. It can be observed that apart from pipe P22, P23 and P24, the pipes having the highest flows are pipe P12, P7 and P4 having flows of 87.93, 49.61 and 43.84 liters per second respectively. The lowest flows are from pipes P13, P10 and P16 having flows of 0.70, 1.57 and 8.64 liters per second respectively. It can be seen from Figure 2 that these pipes are the most farthest from any of the sources in the network. The negative signs in column 4 simply indicate a flow from a junction of lower elevation to one with a higher elevation. The velocities in the pipes have all conformed to the minimum design criterion of 1.5m/s stipulated for Primary distribution pipes. The last column shows values for head losses in the pipes (Tahal, 1995). The EPANET software employs the use of the numerous head loss formulae including Darcy- Weisbach and Hazen Williams. The former was chosen for this analysis. Table 2 gives values for water requirements and Pressure Heads at the nodes of the distribution system.

Table 2: Water requirements and pressure heads at nodes

Node I.D.	Demand(LPS)	Head (m)	Elevation	Pressure (m)
Junction J1	40.00	350.59	325.70	24.89
Junction J2	20.00	350.69	325.50	25.19
Junction J3	10.00	350.88	323.00	27.88
Junction J4	10.00	351.39	331.80	19.59
Junction J5	20.00	351.30	333.60	17.70
Junction J6	15.00	353.15	332.40	20.75
Junction J7	15.00	352.03	333.30	18.73
Junction J8	55.00	356.01	336.00	20.01
Junction J9	50.00	350.47	335.50	14.97
Junction J10	35.00	350.47	334.00	16.47
Junction J11	20.00	350.47	335.00	15.47
Junction J12	25.00	350.78	336.70	14.08
Junction J13	25.00	350.88	331.80	19.08
Junction J14	35.00	351.11	327.80	23.31
Junction J15	45.00	352.34	334.00	18.34
Reservoir 1	120.19	352.42	332.42	20.00
Reservoir 2	170.48	353.30	333.30	20.00
Reservoir 3	129.33	356.10	336.10	20.00

The demand was distributed subjectively at the discretion of the designer with Junctions 3 and 4 obtaining the lowest demands of 10LPS each due to their lesser proximity to other nodes. The highest demand was given to Junctions 8 and 9 with flows of 50 and 55 litres per second respectively considering their further distance to other junctions. The reservoirs were located at distances such that there would be a relatively even distribution in the system. Reservoirs 1, 2 and 3 were located at Junctions J15, J6 and J8 respectively. Special consideration was also given not to locate any of the three reservoirs near Junction J2 because a reservoir supplying Zone 1 was located not more than 200m away from the Junction. If water is to be pumped daily into the three reservoirs and allowed to flow into the network by gravity, the daily water requirement for Reservoirs 1, 2 and 3 would be 10,384,416litres (2,284,252gallons), 14,729,472litres (3,240,031gallons) and 11,174,112litres (2,457,961gallons) respectively. The pressure head of the reservoirs was obtained as 20m during analysis because any value less than this would yield a pressure head less than 12m in the system. This was not acceptable since Tahal Consultants (1995) suggested a minimum of 12m pressure head in the network system. The rationale behind this value is that high rise structures with heights below 12m can access water from the system without the need for personal booster pumps.

2. Conclusion

The zones currently with service reservoirs are zones 1 and 2. Each of the reservoirs has capacities of 1,000,000 gallons each. The Phase 2 of the Maiduguri water supply scheme which was designed in 1995 for Zones 3,4 and 5 of Maiduguri was never achieved till this date. A population of 357,075 people and a daily demand of 420LPS (36,288,000LPD or 7,982,244 gallons per day) would have to be satisfied in line with the Sustainable Development Goals. In order to construct overhead tanks similar to those provided for zones 1 and 2 (i.e 1,000,000 gallons each), Reservoir 1 will require 2 tanks and a smaller one to contain about 300,000 gallons. Reservoir 2 will require 3 tanks and a smaller one containing about 250,000 gallons. Lastly, Reservoir 3 will require 2 tanks and another to contain about 500,000 gallons. However, if a more economic approach is required for instance, the 24hr efficient water supply requirement could be reviewed to 7 hours. Consequently, the reservoir capacity would drop by 29% which in turn would drop the 7,982,244 million gallons to 231,491million gallons a day. This decision would maintain the criteria for sizing of pipes, required pressure head at the nodes and the maximum velocity of flow in the distribution system.

It is therefore recommended that:

3. The pumping time for the water into the reservoirs in zone 3 should not exceed 7 hours to limit the magnitude of cost for constructing overhead tanks
4. Reservoir 1 should have a capacity of about 700,000 gallons, 1,000,000 gallons for Reservoir 2 and 800,000 gallons for Reservoir 3.
5. Relevant agencies should inform the residents of the zone about the time designated for pumping the water.
6. Assessment of the available surface and groundwater should be made in order to determine its sufficiency for the projected population.
7. Residents should be advised to have water harvesting basins or reservoirs to cater for the remaining 17 hours without water supply.

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