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Assessment of Water Distribution Network of National Water Resources Institute Mando Using WaterCAD

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ABSTRACT
The water supply distribution network in National Water Resources Institute, Kaduna does not meet the demand of the institute, partly due to inadequacy of water supply into the distribution system as well as inefficient distribution system, improper connections in the pipes supplying water to the main overhead tank and burst pipes resulting in water loss due to leakages. In this study, Water CAD was used in the assessment of the existing water distribution system of National Water Resources Institute, Kaduna. It is aimed at analyzing water distribution system in the study area, the adequacy of the existing tanks and the main causes of the frequent water problems in the study area. Quantum Geographic Information System (QGIS) software was used to develop the Topographical map of the Institute in order to obtain the spatial information needed for the analysis. An extended period simulation was carried out to determine hydraulic parameters such as pressure, velocity and flow rate. The results of the analysis in the network revealed that the pressure range is from 8.82m to 17.35m, the flow range is from 0.31 l/s to 12.41 l/s and the velocity range is from 0.07m/s to 3.98m/s. This result shows that the pressure, flow and velocity are adequate enough to provide water in the Institute. The main overhead tank of the Institute has a capacity of 210000 litres and existing water demand of the Institute is 167220 litres per day. Finally, some of the reasons for the inefficiency in water delivery from the assessment carried out were the inability to fill up the main overhead tank to its maximum capacity in adequate time, inefficient submersible pumps, improper connections, pipe bursts and power outages.

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INTRODUCTION

Water is one of the most important commodities that all living things need to survive. Its purpose in Commercial, agricultural or irrigation, transportation, recreation, and, of course, general domestic use, makes it a valuable commodity with no other alternatives (Alkali et al., 2017). Therefore, it is necessary to pay close attention to how this water is being conveyed to consumers. A water distribution system is 'a system that supplies water with good quality, adequate quantities and at sufficient pressure to meet system requirements to the users' (Terlumun & Robert , 2019). The water distribution system consists of pipes, valves, hydrants and appurtenances used for water distribution, elevated tanks and reservoir used for fire protection and pressure equalization, and pump discharges and meters (Izinyon & Anyata, 2009). One of the most predominant factors affecting the performance of an existing network is population growth and its associated demand, which may require for complete reticulation or rehabilitation of the existing system. In assessing the efficiency of a water distribution system, the design forms an integral part of the water supply system which

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contributes significantly to curbing expenditures incurred during procurement and construction, hence the need for a systematic design to achieve optimum system performance (Dhumal et al., 2018). In this case, effective water supply is very important when designing new distribution networks or expanding and strengthening existing ones (Izinyon & Anyata, 2009). The water distribution network is an important part of any water supply system and accounts for up to 80% of the total cost of the system (Kleiner & Rajani, 2000). As a result, poor design can increase operational and maintenance costs, therefore the need for well-planned, designed and constructed distribution networks cannot be over emphasized. It should be constructed and operated to minimize the possibility of water pollution after entering the system (Izinyon & Anyata, 2009).

A hydraulic analysis of a water distribution network is required to determine the pressure contours and flow pattern of the system (Sincero & Sincero, 1996) and it involves determining the flow rate and head loss in each pipe and pressure at critical points in the system under different demand conditions (Qasim *et al.,*, 2000). This information can be used by engineers to determine if the system can meet the designed requirements. Various methods have been used to analyze distribution networks, but the oldest and most widely used is the Hardy Cross method, the procedure of which is iterative, tedious and time consuming hence the resort in recent times is the use of computer aided modelling of a water distribution network (Izinyon & Anyata, 2009). A good water distribution network must meet the following requirements; the water quality in the system must not deteriorate, all consumers need to receive enough water at the desired pressure, design and layout must be economical, the system must meet fire requirements, it should be easy to maintain, continue to supply the right quality and quantity of water while minimizing service interruptions.

LITERATURE REVIEW

Water Demand

The water supplied by the distribution system has two main functions. One of them is to satisfy consumer demand, which is the flow in litres per minute needed to provide daily supply for residential, institutions and municipal services and the other is to maintain a proper and consistent supply for fire protection. The determination of consumer demands involves assessing the utilization of water based on the three levels of usage below (Hickey, 2008).

i. The average daily demand reflects the total amount of water used per day, this number varies greatly by state and region. In 2003, the American Water Works Association (AWWA) estimated this figure to 534 litres per capita per day on average.
ii. The maximum daily consumption reflects the day with the highest consumption within the year. The AWWA reported that for any community, this figure is approximately 150% of the average daily demand. The maximum daily consumption is usually reached during the summer months or in the periods of peak demand for industrial use.
iii. The instantaneous flow demand represents the two peak periods of the day between 7a.m to 9a.m and between 5p.m and 7p.m. when consumption is at peak. During these periods, the demand can reach 225% of the average daily use. These figures must be predicted so that the amount of water delivered to the distribution system and the pressure at any particular point will meet the system requirements. Municipal water supplies must always be able to provide the required fire flow through properly placed fire hydrants to address potential fire hazards. The decision for a public water supply to provide fire flows can have significant impact on the design and operation of the system. Large amounts of water are needed to control, contain, and extinguish fires in structures. These quantities often greatly exceed consumers demand. This is the main reason why many settlements with a population of less

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than 5,000 do not have fire hydrants (Hickey, 2008).

The amount of water needed for fire suppression varies across the municipality depending on the condition of the building and the occupants. Therefore, water demand for fire protection must be determined at different locations throughout the municipality. The locations are usually selected by the Insurance Services Office (ISO) for the purposes of insurance rating. According to ISO, the minimum reliable amount of water required is 946 l/m in 2 hours (Hickey, 2008).

Water Distribution Systems

A water distribution system is a network of storage tanks, valves, pumps and pipes that transport treated water to consumers, which has gone through all the processes in a water treatment plant. When designing a distribution system, it is necessary to consider the projected design life of the system and projected population of the area, the relationship between average and peak demand and the allowable system pressure and velocity of flow. It is also necessary to determine all design flows that represent the occupied area of the community and any foreseeable expansions. The basic things to consider when choosing a design flow for the system are average daily demand, maximum daily demand, maximum hourly demand and the required fire flow. Dead ends in the system are usually associated with low water pressure and high water age and every effort must be made to eliminate them (ODEQ, 2009). The water flowing through the distribution system comes into contact with a variety of materials, some of which can significantly change the quality of the water supplied to the consumers. Corrosion of water pipes, valves, and accessories can impair the quality of drinking water (US EPA, 2009). Solids settle under low flow conditions which can be suspended under high flow conditions. Disinfectants and water additives react with organic and inorganic substances to form by-products in the water supply, and there is also problem of biofilm formation (US EPA, 2009).

Storage facilities within the distribution system allow the system to meet demand when the treatment facility is idle or unable to produce demand. It is more advantageous to provide several small storage units at different parts of the system rather than to providing the same large capacity to a central point in the system. It is most economical to bring the storage to full capacity at night when there is minimal domestic consumption and other activities and then increase when storage drops to 40 or 50 percent during the day (Hickey, 2008). Storage can improve or balance system pressure and provide a reserve in case of an emergency such as power outage. The amount of water needed to balance water production is 30 to 40 percent of total storage available for water pressure equalization and emergency water supply reserves (Hickey, 2008).

However, water storage adversely affects water quality by creating conditions in which disinfectant residues are lost as the water ages, bacteria re-grow, taste and odour are also generated, and disinfection by-products are formed. Improper mixing in storage facilities can exacerbate the problem of water aging by creating dead zones with older water. Domestic supply is usually supplied from the top 25-30% of storage capacity, after which the control of high performance pumps covers the demand and begins to fill the tank. The remaining 70-75% is usually stored as a dedicated fire storage (Hickey, 2008).

It is recommended to keep the distance between fire hydrants within 100m in crowded high-risk areas and 150m or less in residential areas where the distance between buildings exceeds 15m. It is also advisable to install fire hydrants at all intersections and dead ends. Generally, it is recommended to install fire hydrants on pipes with a diameter of 152mm or more (Hickey 2008).

Criteria for Performance Evaluation

The performance of the system is measured by the ability of the system to always provide high quality water under proper operating conditions (Coelho, 1997). This performance depends on several criteria. Planning for these systems is very important that need to consider the following factors:

a. Design life of the system

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b. Appropriate advantages of topographic features to reduce energy costs
c. Projected population growth
d. Projected industrial and commercial growth
e. Water consumption data: average daily consumption, per capita consumption and peak flow factors
f. Minimum and maximum acceptable pressures.
g. Storage facilities (Swamee & Sharma, 2008).

Based on the above criteria, design period can be based on projected growth. Alternatively, for a static population such as a rural water community, the design period can be based on the life period of the pipes (Swamee & Sharma, 2008). The Oklahoma Department of Environmental Quality has published guidelines on public drinking water systems that if a community is supplied with groundwater, it needs at least two wells or one reserve water source to ensure adequate water supply. If the city is supplied only with water from groundwater resources, the capacity of the well must be greater than or equal to the design maximum day demand and the design average day demand (ODEQ, 2009).

All pumping stations will have two pumps. In case of the failure of one pump, the other pump should be able to supply water during the peak periods of the day to maintain optimum pressure. All storage tanks should be able to provide enough storage facility to meet the regular average daily demands satisfying peak hourly periods but most importantly fire flow demands at a key location peak hours (Salvato, 1992). Generally, the peak hourly flow factors are 3 to 6 times the average daily flows (Haestad*et al.*, 2003). Also the maximum design variation in the storage levels should not vary more than 9 m to maintain the required pressures. In case the distribution system does not provide fire protection, then it should have storage capacity of 24 hours and must be able to maintain a pressure of at least 25 psi throughout the distribution system (ODEQ, 2009). As per the Insurance Services Office (ISO), towns having fire class greater than 8 should be able to provide a flow of 946 l/m at peak daily demand at a pressure of at least 20 psi (ISO Mitigation Online, 2009). Also dead ends should be minimized by looping them to the main network system (ODEQ, 2009). It is desirable to install a fire hydrant or flushing device at the dead end to avoid the problem of water pollution due to stagnation (ODEQ, 2009).

Principles of Hydraulic Modeling

In hydraulic simulation modeling a distribution network is considered to be one in which all elements are connected to each other, every element is influenced by its neighbours, and each element is consistent with the condition of all other elements. These conditions are mainly controlled by: Law of Conservation of Mass and Law of Conservation of Energy. The total mass of water entering the system must be equal to the total mass of water exiting the system, and the total flow at a particular node must be zero. The principle of conservation of energy is mainly determined by the Bernoulli's equation, which states that the difference in the energy between any two points should be the same regardless of the path taken (Haestad *et al.*, 2003).

A typical network of hydraulic models consists of the following components:

• Nodes linking the pipes
• Pipes
• Storage tanks
• Reservoirs
• Pumps
• Additional appurtenances like valves (Haestad *et al.*, 2003; Rossman, 2000).

The junctions or nodes represent points having particular base demands. Tanks are those points in model which can have a specific storage capacity that varies with time. Reservoirs in a hydraulic model are assumed to be an infinite source of water (Haestad *et al.,* 2003; Rossman 2000). Pumps are energy devices which provide pressure and head to the water. The graph of head vs. flow for a particular pump is called the '*pump curve*'. Generally, there are three parameters that define the pump operation; the shut off head, the design point and the maximum point. The system curve is an important curve necessary to decide the best operating point of pump. The pump

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should be able to overcome the elevations differences, which is dependent on the topography of the system. The head added on the pump to overcome these differences is called the static head. Friction and minor losses also affect the discharge through the pump. When these losses are added to the static head for different discharge rates, the plot obtained is called system head curve (Haestad *et al.*,, 2003).

Modeling using WaterCAD

WaterCAD is a hydraulic simulation software distributed by Bentley Systems. After creating the spatial model, there is need to define the following parameters for each model component:

1. Nodes: Elevations and the base demands
2. Pipes: Pipe diameters, lengths and the friction coefficient factors. By default, WaterCAD considers the pipe material to be ductile iron with a Hazen William friction coefficient factor of 130
3. Tanks: Base Elevation, the minimum and maximum levels, diameter of the tank
4. Pumps: The most important parameter that defines the operation of the pump is the pump curve, other input needed is the elevation of the pump.
5. Reservoir: Elevation
6. After all the parameters needed to run the simulation have been entered into the model, if the simulation runs successfully, then the following solution is obtained.
7. Pressure at every single element in the system
8. Flows at every point of time in the system
9. Velocities in the pipes
10. Levels in the tanks
11. Pump cycles
12. Water age and constituent concentration.
13. Additionally, it has the capability of performing the analysis of the system for the steady state scenarios and for an extended period of any length. The other capabilities of the software include:
14. Evaluate the hydraulics for different demands at a single node with varying time patterns
15. Solve for different frictional head losses using Hazen-William, Darcy-Weisbach or the Chezy-Manning equations
16. "Can determine immediate inefficiencies in the system" (Haestad *et al.,* 2003)
17. Determine fire flow capacities for hydrants
18. Model tanks, including those which are not circular
19. Model various valve operations
20. Provides control based operations
21. Perform energy cost calculations
22. Model fire sprinklers, irrigation systems, leakages and pressure dependent demands at any particular node (Haestad *et al.,* 2003).

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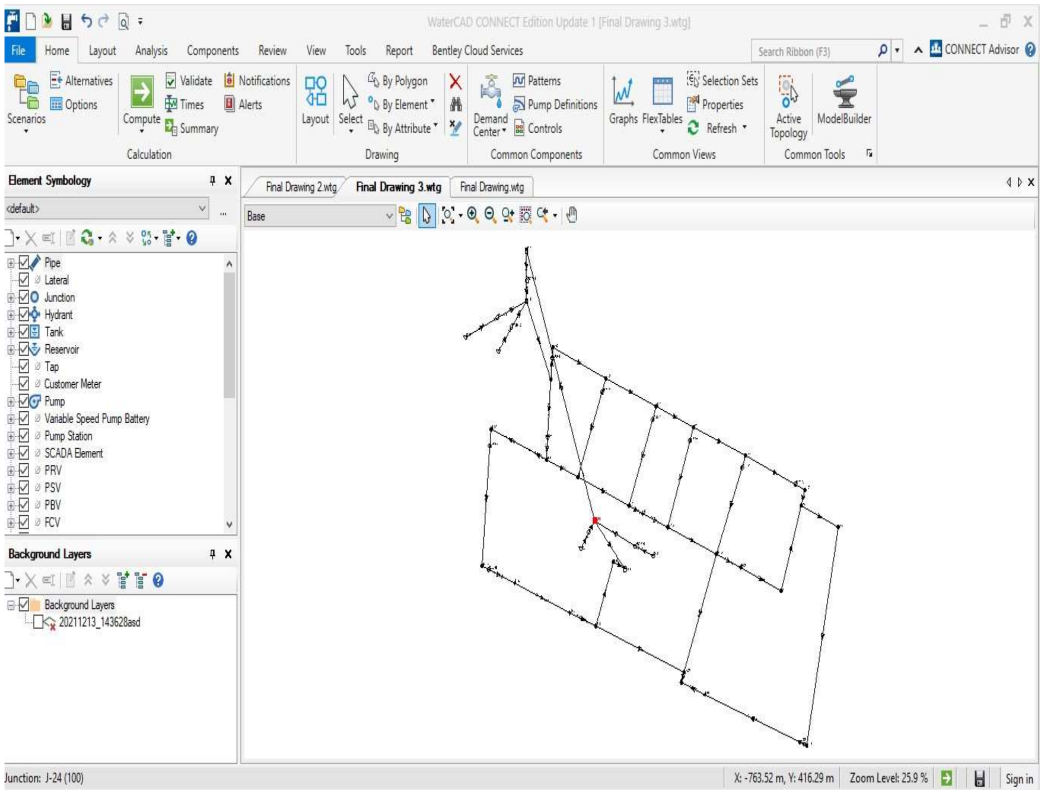

Figure1: Shows the user interface for WaterCAD.

Some few literatures on research conducted on the performance of water distribution network using different approaches, includes: Elsheikh et. al., (2013) used WaterCAD software to investigate the water distribution network in Tanta, Egypt, through a ten-year design period to determine whether it was feasible to expand the current distribution network with a new pipe network to service additional and new areas. WaterCAD software is used to design and optimize distribution networks, considering the aging of pipes and significant problems with the quality of the water supplied. Various WaterCAD tools such as Darwin Designer and Pipe Catalog Tools have proven to be very effective in overall planning and optimization of distribution networks. The research gap is the location of the study area where the assessment was carried out.

Harding, (2007) evaluated the water supply system of the Mae La temporary shelter in Thailand using GIS technology. Approximately 7,117 homes were randomly selected in a survey using Google Earth and site visits. ArcGIS and Epanet were used to estimate daily drinking water per household. According to his research, the flow rate predicted by the Epanet model is highly dependent on the elevation of the points in the distribution network infrastructure and is difficult to determine accurately. Thus, although the end result shows that one out of four households were not adequately served. However, the research revealed that the majority of residents in Mae La have sufficient access to water.

Arunkumar & Mariappan, (2011) evaluated and assessed public water supply system using EPANET 2.0 software for the systematic planning and operation of distribution system over a 30-year design period. Hydraulic model was built and run using EPANET 2.0 for the "A" Zone, Thirumullaivoyal, Avadi city of Tamilnadu, India. The model was tested and run with a 24-hour supply and a 6-hour intermittent

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supply and the resulting pressure at various nodes were checked. The pressure of the intermittent supply was almost doubled with a value of 24.03m of the pressure observed with 24-hour supply of 12.02m, by providing a pressure break and pressure release valve, uniform pressure was achieved throughout the model. EPANET 2.0 software was efficiently used to compare 24-hour and intermittent 6-hour supplies for public water systems.

Ayanshola & Sule, (2006) evaluated the flow pressure in selected zones of the water supply network at the city of Ilorin in Kwara, Nigeria. Pipe network analysis was performed based on the demand for 10 cases using the service reservoir location and the actual production volume for distribution to the three zones within the study area. The maximum and minimum nodal pressures obtained were compared to the recommended values for satisfactory performance of the water supply network. The result shows that poor pipeline water distribution due to inadequate flow and pressure can impede equitable access to the services by households.

Mohapatra et al., (2012) investigated the efficiency of water supply systems using EPANET and ArcGIS software. They conducted an investigation on leakages in continuous and intermittent water supply systems at Untkhana in Nagpur, India and the simulated results were reviewed in the continuous water supply system, higher pressure was observed with a value less than 12.0 m. EPANET Software was used successfully for the simulation of both intermittent and continuous supply, which concluded that avoidance of direct tapping to transmission mains and immediate repair of leakages can improve the system.

METHODOLOGY

Study Area

The National Water Resources Institute (NWRI) Kaduna was established in 1985 and mandated by law with the responsibilities of carrying out researches in providing solutions to existing water related problems within and outside the shores of Nigeria (NWRI, 2022). It functions include research, management, well/borehole drilling, maintenance distribution as well as repair of infrastructure to aid effective utilization of water. The Institute is located along Mando road Kaduna, Nigeria and it lies at latitude 10°34´58"N and Longitude 7°25´11"E in North Western Nigeria. The topography of the area is a differential one, i.e. one side of the area is relatively flat, while the other side is slightly elevated. The Institute has a residential population of about 929.

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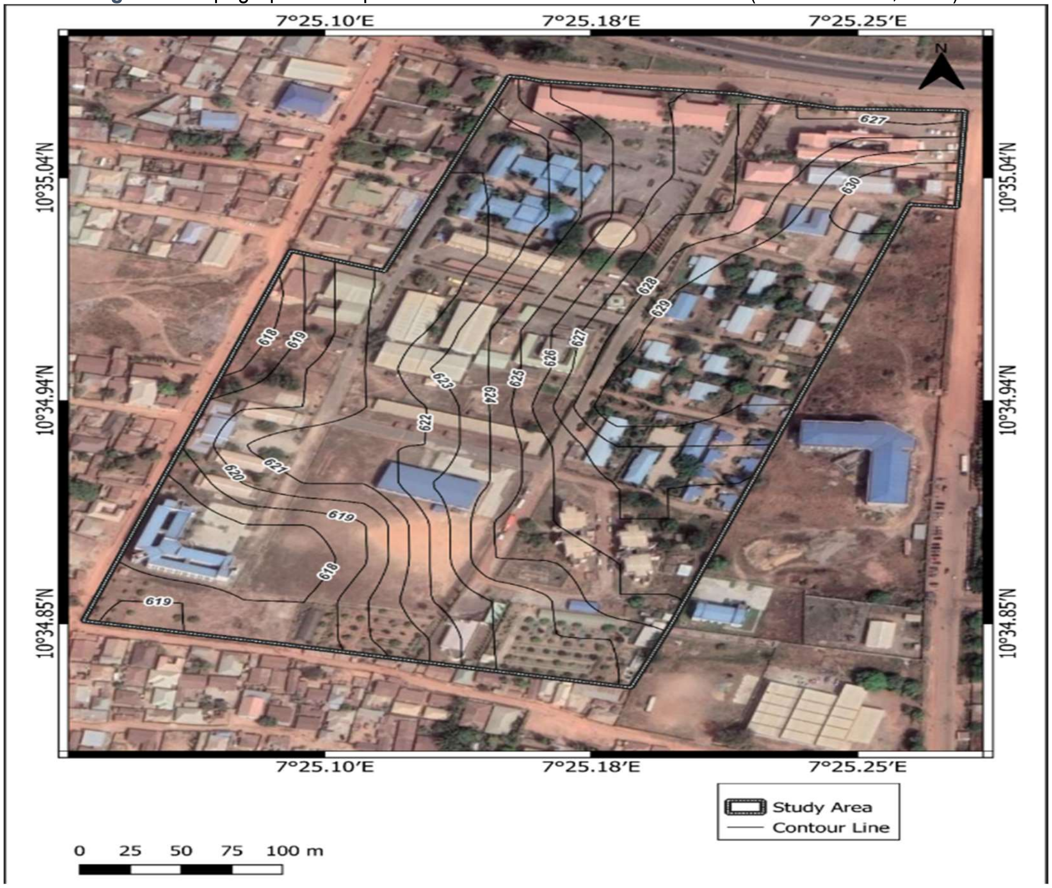
Figure 2: Topographical Map of National Water Resources Institute (Source: QGIS, 2022)



Materials

The materials used for this study include: topographical map, WaterCAD software, Google Earth, Quantum Geographic Information System (QGIS).

Data Collection

Several field visits to the National Water Resources Institute were conducted. The following information were obtained from the records of the various units and departments of the Institute.

a) Design Population: The population information was obtained from the Facility Management Unit (FMU), Human Resources Department and student's affairs division. The design population for the staff quarters was obtained by taking an average of Seven (7) persons per house, two (2) persons per boy's quarters and one (1) person per room for the guest house, hostels and water suites. The population of National Water Resources Institute is summarized in the table below.

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Table 1: Population of National Water Resources Institute

S/N	Category	Average no. of person allocated	Total population
1	Staff Quarters (28 Flats)	7	196
2	Boys Quarters (16 Rooms)	2	32
3	Guest House (10 Rooms)	1	10
4	New Hostel (116 Rooms)	1	116
5	Old Hostel (54 Rooms)	1	54
6	Water Suites (60 Rooms)	1	60
7	Staff	-	191
8	Students	-	270
	Total	-	929

b) Quantum Geographic Information System (QGIS) was used to obtain the topographical map of the study area.

c) Water Reservoir: The main sources of water supply at the institute is from ground water. It was discovered that four existing boreholes are used to pump water to the main overhead tank and later distribute to the water network. Boreholes nos. 1 and 2 were about 50m deep, located close to the main overhead tank and were connected directly to the main overhead tank while Boreholes nos.3 and 4 were about 60m and 100m deep respectively and were connected to 3 ground tanks which uses pump to lift the water to the overhead tank with a static water level of about 9.8m.

d) Storage tank: The main storage tank was constructed in 2016 using steel with a reinforced concrete base. It has a capacity of holding 210m^3 (210000 litres) of water when in full capacity and it has a height of 11m.

e) Pumps: The submersible pumps in Boreholes no. 1 and 2 have a rating of 0.5 HP, while that of Boreholes no. 3 and 4 have a rating of 1HP and 2HP respectively. Pumps 1, 2 and 3 were placed at a depth of 20m below ground level while pump 4 was placed at a depth of 40m.

Steps in Using WaterCAD

This software offers various options for modeling your network. If drawings and dimensions are available, the user can physically draw the network. Alternatively, you can import files from WaterCAD database, Sub models and EPAnet. The existing water distribution layout map of the institute was used as background layers on the WaterCAD platform, where the pipes are linked with nodes at each junction on the map together with the available parameters.

The following steps were carried out to assess the existing water distribution network of the Institute:

i. Create a new hydraulic model file.
ii. Layout the network (a background file was used to layout the pipe network).
iii. Enter and modify data (Data was entered and modified using the property editor and flex tables).
iv. Create Demand Patterns.
v. Run an Extended Period Simulation (Bentley WaterGEMS, 2015).

Assigning Distribution Network Parameters

After skeletonizing your network on the WaterCAD platform, the next step is to assign network parameters. The network parameters include pipe length, pipe diameter, roughness coefficients (Hazen-Williams or Darcy-Welsbach), Nodes number, and Nodal elevation. These are the basic network parameters that will be the basis for future simulations, depending on the flow you are simulating.

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Assigning Roughness Coefficients to Pipelines

Hazen-William roughness factors will be used to incorporate frictional losses. WaterCAD has an engineering library that stores different frictional factors for different pipe materials, the material of the pipe can be selected in the pipe property editor. By default, WaterCAD considers pipeline to be a new ductile iron pipe. In general, pipes made of materials such as steel, PVC and asbestos cement are less prone to deposition and corrosion than cast iron pipes. Figure 3 shows a table of Hazen William's coefficient of friction, these are categorized by line size, age, and degree of attack. Attacks on pipes are defined as pipe corrosion, and the higher the "C" factor, the smoother and load-bearing capacity of the pipe (Haestad *et al.,* 2003). This table was used to assign the respective friction factors.

Table 2: C-factor values for Different line sizes

	C-factor Values for Discrete Pipe Diameters					
Type of Pipe	1.0 in. (2.5 cm)	3.0 in. (7.6 cm)	6.0 in. (15.2 cm)	12 in. (30 cm)	24 in. (61 cm)	48 in. (122 cm)
Uncoated cast iron - smooth and new		121	125	130	132	134
Coated cast iron - smooth and new		129	133	138	140	141
30 years old						
Trend 1 - slight attack		100	106	112	117	120
Trend 2 - moderate attack		83	90	97	102	107
Trend 3 - appreciable attack		59	70	78	83	89
Trend 4 - severe attack		41	50	58	66	73
60 years old						
Trend 1 - slight attack		90	97	102	107	112
Trend 2 - moderate attack		69	79	85	92	96
Trend 3 - appreciable attack		49	58	66	72	78
Trend 4 - severe attack		30	39	48	56	62
100 years old						
Trend 1 - slight attack		81	89	95	100	104
Trend 2 - moderate attack		61	70	78	83	89
Trend 3 - appreciable attack		40	49	57	64	71
Trend 4 - severe attack		21	30	39	46	54
Miscellaneous						
Newly scraped mains		109	116	121	125	127
Newly brushed mains		97	104	108	112	115
Coated spun iron - smooth and new		137	142	145	148	148
Old - take as coated cast iron of same age						
Galvanized iron - smooth and new	120	129	133			
Wrought iron - smooth and new	129	137	142			
Coated steel - smooth and new	129	137	142	145	148	148
Uncoated steel - smooth and new	134	142	145	147	150	150

(Source: Haestad *et al.,* 2003)

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Table 2: (cont): C-factor values for Different line sizes **(Source: Haestad *et al.,* 2003)**

Type of Pipe	C-factor Values for Discrete Pipe Diameters					
	1.0 in. (2.5 cm)	3.0 in. (7.6 cm)	6.0 in. (15.2 cm)	12 in. (30 cm)	24 in. (61 cm)	48 in. (122 cm)
Coated asbestos cement - clean		147	149	150	152	
Uncoated asbestos cement - clean		142	145	147	150	
Spun cement-lined and spun bitumen- lined - clean		147	149	150	152	153
Smooth pipe (including lead, brass, copper, polyethylene, and PVC) - clean	140	147	149	150	152	153
PVC wavy - clean	134	142	145	147	150	150
Concrete - Scobey						
Class 1 - Cs = 0.27; clean		69	79	84	90	95
Class 2 - Cs = 0.31; clean		95	102	106	110	113
Class 3 - Cs = 0.345; clean		109	116	121	125	127
Class 4 - Cs = 0.37; clean		121	125	130	132	134
Best - Cs = 0.40; clean		129	133	138	140	141
Tate relined pipes - clean		109	116	121	125	127
Prestressed concrete pipes - clean				147	150	150

Assigning Water Demands to each Node

Population Demand: The population demand at each node was obtained by multiplying the population of that particular node by the per capita demand of 180 L/C/D from the Federal Ministry of Water Resources manual since the study area falls under the category of urban settlement (Saminu *et al.*,, 2013).

Fire Demand: Since a large amount of water is required to extinguish a fire, therefore provision is made in the water work to supply sufficient quantity of water or keep as reserve in the water mains for this purpose. When analyzing total water demand, it is expected that a supply of approximately 10% will be provided for the fire demand. In this case, 10% of the population demand was added as fire demand (Lingkungan, 2012).

Minor Losses: A provision of 5% was made for minor losses. This is to take care of losses at fittings, valves and bends (Adeniran & Oyelowo, 2013).

Unaccounted for Water (UFW): Unaccounted for water consists basically of two components, Water loss from the system and water that is being used but not well documented. It is important to understand that UFW does not equate to "leaks." Water can be "unaccounted for" because of faulty meters and use for purposes that are not metered, such as gardening, flushing, draw offs by water tanker and blow offs for water quality reasons. The average amount of unaccounted for water as a percent of water usage is 12% worldwide, Since the study area for this work is in a developing country, 15% is allowed for as UFW (Adeniran & Oyelowo, 2013).

Hydraulic Modeling in WaterCAD

This section describes how all the model parameters necessary to run the model were set.

Setting Pump Data

The submersible pumps in Boreholes no. 1 and 2 have a rating of 0.5hp, maximum head of 50m and maximum discharge of 5.1 m^3/h. The pump in borehole no. 3 has a rating of 1hp, maximum head of 79m and maximum discharge of 5.1 m^3/h. The pump in borehole no. 4 has a rating of 2hp, maximum head of 150m and maximum discharge of 5.1 m^3/h. The lifting pump has a rating of 7.5hp, maximum head of 64m and

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maximum discharge of 7.5 m^3/h. Pumps 1, 2 and 3 are placed at a depth of 20m below ground level while pump 4 is placed at a depth of 40m. The most important parameter simulating the operation of the pumps is the pump curve. In components option, the user defines the pump curves. Then the user can create more than one pump curve. Each pump curve has a unique ID. The option of design point (1 point) data curve was selected and the design pressure head and discharge were entered in the table. The graph of the curve can be previewed in the window below.

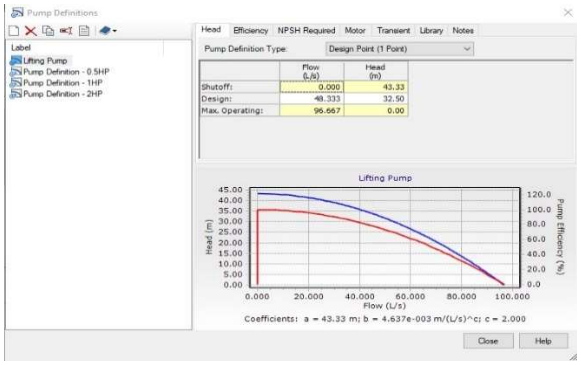

Figure3: Pump curve interface in WaterCAD

Assigning Demand Patterns

The components tab has an option called 'Patterns' which opens the Pattern Manager window. The user can use this pattern manager to create water usage patterns based on daily, weekly and monthly use. For the maximum hourly demand, a multiplier of 5 was used. Generally, the peak hourly flow is 3-6 times the average daily flow (Haestad *et al.,* 2003). The peak hours were considered to be 6 am to 9 am in the morning and 5 pm to 8 pm in the evening. A demand pattern for the offices was created considering the number of staffs and students in the institute and then another demand pattern for staff quarters, hostels and Water Suites was also created. The offices was assumed to be open from 8 am to 4 pm on weekdays.

Peak day factor has a factor of 1.5 for population over 10,000 and 2 for population below 2000, while Peak hour factor has a factor of 2 for population over 10,000 and 5 for population below 2000 (Swamee & Sharma, 2008).

Operating on Rules

Information obtained after discussion with Mr. Abdullahi, the pump operator, was that the main overhead tank of the institute has never achieved its maximum capacity since it was constructed in 2016 as a result of unstable power supply and operation conditions. Therefore, a set of binding conditions that will control the pump operation was set to find out how long it will take to fill up the tank.

The rules written to trigger the pumps on and off were:

IF T-1 Percent full <= 30.0% THEN PMP-1 Pump Status = On
IF T-1 Percent full = 100.0% THEN PMP-1 Pump Status = Off
IF T-2 Percent full <= 30.0% THEN PMP-4 Pump Status = On
IF T-2 Percent full = 100.0% THEN PMP-4 Pump Status = Off
IF T-2 Percent full <= 30.0% THEN PMP-5 Pump Status = On
IF T-2 Percent full = 100.0% THEN PMP-5 Pump Status = Off
IF T-1 Percent full <= 30.0% THEN PMP-3 Pump Status = On
IF T-1 Percent full = 100.0% THEN PMP-3 Pump Status = Off
IF T-1 Percent full <= 30.0% THEN PMP-2 Pump Status = On
IF T-1 Percent full = 100.0% THEN PMP-2 Pump Status = Off

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IF T-3 Percent full <= 30.0% THEN PMP-4 Pump Status = On
IF T-3 Percent full = 100.0% THEN PMP-4 Pump Status = Off
IF T-3 Percent full <= 30.0% THEN PMP-5 Pump Status = On
IF T-3 Percent full = 100.0% THEN PMP-5 Pump Status = Off
These rules were entered using the Controls options under the Components tab.

Head loss Equations

Hazen-Williams Formula

$h = \frac{kL}{D^{1.16}}\left(\frac{V}{C}\right)^{1.85}$ equation 1

Where:
h = Head loss
D = Diameter (in ft. or m)
V = Velocity (in fps or m/s)
C = Hazen-Williams C-Factor
L = Length (in ft. or m)
k = 6.79 for V in m/s and D in m.

Darcy Weisbach Formula

$h = f\frac{LV^2}{D2g}$ equation 2

Where:
h = Head loss
f = Friction factor
L = Length
V = Velocity
D = Diameter
g = Acceleration due to gravity.

Manning's Equation

$V = C_0 R^{2/3}(h/L)^{1/2}/n$ equation 3

Where:
C_0 = 1.49 for English units and 1.0 for metric units
V = Velocity (fps or m/s)
R = Hydraulic radius (ft. or m)
h = Head loss (ft. or m)
L = Length (ft. or m), n = Manning's roughness coefficient.

WATER DEMAND RESULTS

The analysis of nodal demand at each node is given in the table below.

Table 3: The analysis of nodal demand at each node

Node	Name	Population	Lpcd	Demand Daily l/day	Demand l/s	Fire Demand 10%	Minor Losses 5%	UFW 15%	Total Nodal draw off l/s
1	Water suites	60	180	10800	0.125	0.013	0.006	0.019	0.163
2	3 Bedroom Semidetached (2 no.)	14	180	2520	0.029	0.003	0.002	0.005	0.039
3	3 Bedroom Detached (3 no.)	21	180	3780	0.044	0.004	0.002	0.006	0.056
4	3 Bedroom Detached (3 no.)	21	180	3780	0.044	0.004	0.002	0.006	0.056
5	Boys Quarters	32	180	5760	0.067	0.006	0.003	0.010	0.086
6	3 Bedroom Terrace (6 no.)	42	180	7560	0.100	0.010	0.010	0.020	0.130
7	-	-	-	-	-	-	-	-	-
Node	Name	Population	Lpcd	Demand Daily l/day	Demand l/s	Fire Demand 10%	Minor Losses 5%	UFW 15%	Total Nodal draw off l/s
8	-	-	-	-	-	-	-	-	-
9	-	-	-	-	-	-	-	-	-
10	-	-	-	-	-	-	-	-	-
11	Admin Block/Library	70	180	12600	0.15	0.02	0.01	0.02	0.200
12	Facility Management Unit	10	180	1800	0.020	0.000	0.000	0.000	0.020
13	3 Bedroom Converted (3 no.)	21	180	3780	0.040	0.004	0.002	0.006	0.100
14	3 Bedroom Converted (3 no.)	21	180	3780	0.040	0.004	0.002	0.006	0.100
15	4 Bedroom/Guest House	24	180	4320	0.10	0.004	0.002	0.006	0.100
16	3 Bedroom Terrace (6 no.)	42	180	7560	0.100	0.010	0.010	0.020	0.130

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17	UNESCO	20	180	3600	0.040	0.002	0.001	0.001	0.100
18	Offices/Lecture Hall/Training Dept./Mosque	270	180	48600	0.56	0.056	0.028	0.084	0.730
19	Workshop/RTS/JICA	86	180	15480	0.180	0.018	0.010	0.030	0.240
20	Modelling Centre	5	180	900	0.010	0.001	0.001	0.002	0.014
21	Hostel	54	180	9720	0.110	0.010	0.010	0.020	0.130
22	New Hostel	116	180	20880	0.240	0.024	0.012	0.036	0.310

Table4: Flow and Velocity in Pipes

Label	**Flow (L/s)**	**Velocity (m/s)**	**Headloss Gradient (m/m)**
P-1	6.079	0.77	0.006
P-2	6.063	0.77	0.006
P-3	3.498	0.45	0.002
P-4	2.775	0.35	0.001
P-5	2.410	0.31	0.001
P-6	1.420	0.18	0.000
P-7	1.420	0.18	0.000
P-8	2.108	0.27	0.001
P-9	2.108	0.68	0.008
P-11	-0.548	0.07	0.000
P-12	1.937	0.25	0.001
P-14	2.840	0.36	0.001
P-16	-0.689	0.09	0.000
P-18	2.524	0.32	0.001
P-19	0.683	0.09	0.000
P-20	0.313	0.04	0.000
P-21	0.939	0.12	0.000
P-23	-4.861	0.62	0.004
P-24	-1.932	0.25	0.001
P-26	6.580	0.84	0.007
P-27	2.938	0.94	0.014
P-28	-10.471	1.33	0.016
P-29	12.409	3.98	0.207
P-30	-41.704	9.44	1.263
P-31	-41.704	9.44	1.263
P-32	1.624	1.42	0.055
P-33	1.624	1.42	0.055
P-34	1.798	1.58	0.067
P-35	1.798	1.58	0.067
Label	**Flow (L/s)**	**Velocity (m/s)**	**Headloss Gradient (m/m)**
P-36	-10.471	1.33	0.016
P-37	2.426	2.13	0.117

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P-38	2.426	2.13	0.117
P-39	2.112	1.85	0.090
P-40	2.112	1.85	0.090
P-41	2.893	2.54	0.162
P-42	1.644	1.44	0.057
P-17(1)	6.263	0.80	0.006
P-17(2)	6.263	0.80	0.006
P-10(1)	-6.780	0.86	0.007
P-10(2)	-6.780	0.86	0.007
P-13(1)	2.580	0.33	0.001
P-13(2)	2.580	0.33	0.001
P-15(1)	0.789	0.10	0.000
P-15(2)	0.789	0.10	0.000
P-22(1)	5.850	0.74	0.005
P-22(2)(1)	5.850	0.74	0.005
P-22(2)(2)	5.850	0.74	0.005
P-25(1)	-1.984	0.25	0.001
P-25(2)	-1.984	0.25	0.001
P-45	-2.108	0.27	0.001

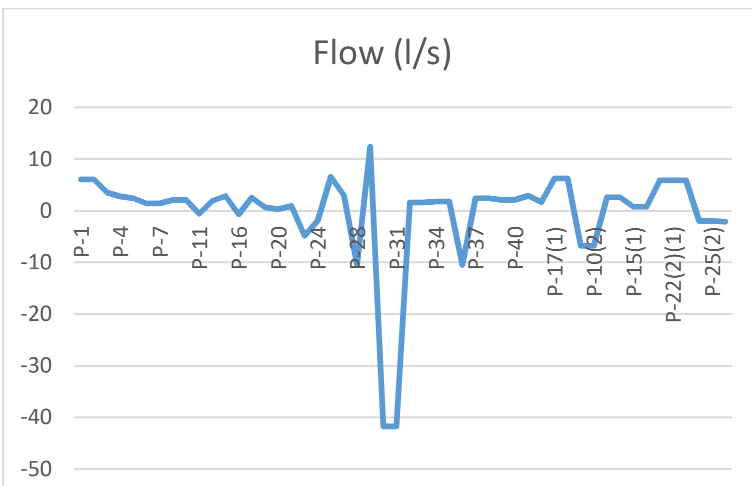

Figure 4: Flow in Pipes

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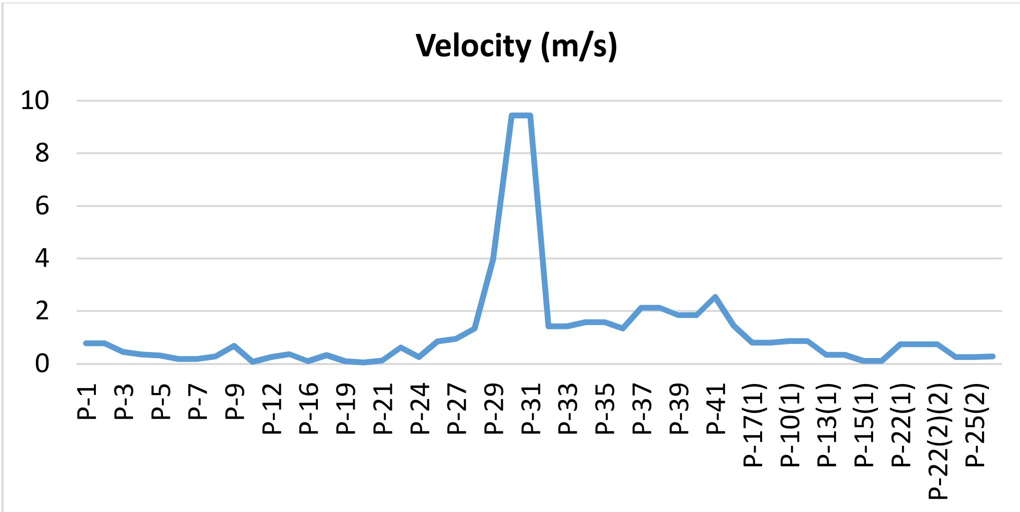

Figure 5: Velocity in Pipes

Network Junction Result

Table 5: Nodal Pressure Heads Results

Label	Elevation (m)	Demand (l/s)	Hydraulic Grade (m)	Pressure (m H2O)
J-1	628.00	0.068	637.90	9.88
J-2	629.00	0.016	637.84	8.82
J-3	628.00	0.040	637.48	9.46
J-4	628.00	0.040	637.33	9.31
J-5	628.00	0.052	637.25	9.23
J-6	627.00	0.052	637.17	10.15
J-7	626.00	0.000	637.14	11.12
J-8	625.00	0.000	637.13	12.11
J-9	625.00	0.000	637.08	12.06
J-10	621.00	0.000	635.53	14.50
Label	**Elevation (m)**	**Demand (l/s)**	**Hydraulic Grade (m)**	**Pressure (m H2O)**
J-11	627.00	0.200	636.83	9.81
J-12	626.50	0.030	637.36	10.84
J-13	626.50	0.040	637.37	10.85
J-14	626.00	0.040	637.32	11.29
J-15	626.00	0.052	637.25	11.22
J-16	625.00	0.052	637.15	12.13
J-17	624.00	0.100	637.14	13.11
J-18	624.00	0.730	636.00	11.97
J-19	622.00	0.240	635.03	13.00
J-20	621.00	0.010	635.46	14.43

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Label	Elevation (m)	Demand (l/s)	Hydraulic Grade (m)	Pressure (m H2O)
J-21	621.00	0.052	635.47	14.44
J-22	619.00	0.124	635.53	16.49
J-23	623.00	0.000	634.11	11.09
J-24	626.00	0.000	643.38	17.35

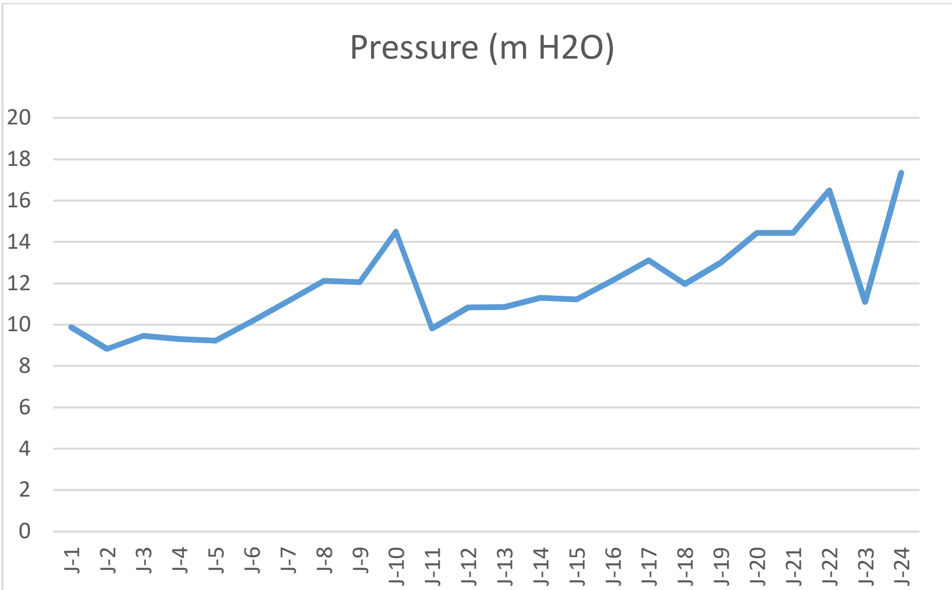

Figure 6. Nodal Pressure Heads Result

DISCUSSION OF RESULTS

Flow and Velocity in Pipes

Design and Layout of water distribution networks in Building Code compliance document G12/ASI sets out acceptable minimum flow rates in pipes at 0.30 l/s while the velocity must not exceed 3.0m/s. (http://www.level.org.nz/water) (Adeniran & Oyelowo, 2013).

Figure 4 shows the graph of flow rate in pipes, the flow rate in the network ranges from 0.31 l/s to 12.41 l/s. The results of the analysis of flow in the network shows that the flows in the network is good with all the pipes having flow rate above 0.3 l/s. However, pipe 30 and pipe 31 having flows of -41.07 l/s are the pipes supplying water from the ground tanks through a 7.5 hp lifting pump to the main overhead tank.

Figure 5 shows the graph of velocities in pipes, the results of the velocity in the network ranges from 0.07m/s to 3.98m/s. The velocity of flow in the network is good. All the pipe velocities are below 3.0 m/s except for pipe 21 coming directly from tank 1 which is having a velocity of 3.98 m/s. The two pipes having velocities of 9.44 m/s are also pipes 30 and 31 supplying water from the ground tank through a 7.5 hp lifting pump to the main overhead tank.

Nodal Pressure Heads Results

Figure 4.4 shows the graph of Nodal pressure heads, most of the buildings in the Institute are not more than 9m in height, and they are mostly two story buildings. From the results obtained, it is observed that all the nodes have pressure heads above the required minimum pressure head of 9m. The result shows that the pressure in the system are generally good.

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Pump Controls and Operations

The analysis was run for 48 hours in order to simulate the water flow in the network. The pump controls were set to switch ON the pumps when the water level in tank 1 is below 30 percent and the pumps are turned OFF when the level of water in tank 1 is 100 percent, the pump status was ON at the start of the simulation. In the first pump circle, it was observed that tank 1 reached its maximum level in 17 hours. In the next pump circle, tank 1 reached its maximum level in 21 hours.

CONCLUSION

The focus of this study is to analyze the existing water distribution system of National Water Resources Institute, Kaduna and to identify deficiencies (if any) that may be present in the system using waterCAD. The hydraulic simulation carried out revealed that the pressure in the water distribution system ranges from 8.82m to 17.35m which is adequate enough to provide water to the Institute. The result of the analysis in the network shows that the flows and velocities in the network are generally good with a flow of not less than 0.3 l/s and a velocity below 3 m/s (Adeniran & Oyelowo, 2013).

At constant power supply, the main tank has the capacity to deliver 210m^3 (210000 litres) of water, while the existing demand of water use per day is 167220 liters per day. This clearly shows that the main tank is also sufficient enough to provide water in the study area.

The water supply distribution in National Water Resources Institute is not currently inadequate. On review of the situation, various factors were identified as affecting the availability of water to end users.

One of the main factors affecting the availability of water in the institute is the inability to fill up the main overhead tank to full capacity. However, a series of pump operation controls were simulated in waterCAD to start the pumps when the level of tank 1 is below 30% full and set to shut off the pumps when tank 1 is 100% full, this takes about 17 hours for the tank to reach its maximum capacity. Tanks should ideally fill up within 6 to 12 hours of pump cycle (Salvato 1992). It was also observed that some overhead tanks situated by the boreholes around the computer centre and the conference centre are connected to the pipe supplying water to the main overhead tank. These overhead tanks by the boreholes have to fill up first before the water is pumped into the main tank by the gate which also delays the time it takes to fill up the main overhead tank.

RECOMMENDATIONS

i. It is recommended that bigger and more efficient pumps should be used in achieving maximum capacity of the main tank.
ii. The overhead tanks by the computer and conference center should be isolated from the pipes supplying water to the main overhead tank so as to achieve maximum tank capacity in a shorter period of time.

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