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WATER DISTRIBUTION NETWORK MODELLING OF A SMALL COMMUNITY USING WATERCAD SIMULATOR

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ABSTRACT

In this study a network model was constructed for the hydraulic analysis and design of a small community (Sakwa) water distribution network in North Eastern geopolitical region of Nigeria using WaterCAD simulator. The analysis included a review of pressures, velocities and head loss gradients under steady state average day demand, maximum day demand conditions, and fire flow under maximum day demand using average day demand of 60 lpcd.The results indicate: that the tower height of the existing storage tank is inadequate and should be increased from 10m to 15m to provide satisfactory service, and that there are no areas of concern with respect to pressure or available fire flow for the proposed service area and also that flow velocities are not excessive while head loss gradients in the network are within acceptable limits. Pipes P-6, P-12, P-15 and P-19 expectedly have relatively low flow velocities due to the low average day demands in small communities and the constraint of minimum commercially available pipe sizes makes design of self cleansing networks in such communities not easily realizable.

KEYWORDS: Network model, WaterCAD simulator, pressure, flow velocity, head loss gradient.

INTRODUCTION

A water distribution network is the means of getting water from the source to the consumer. It serves to convey the water from the water source and treatment works where necessary to the point where it is delivered to the consumer (Hofkes & others, 1981). The distribution system of a water works consists of the pipes, valves, hydrants and appurtenances used for distributing the water, the elevated tank and reservoir used for fire protection and for equalizing pressures and pump discharges and meters (Camp and Tawler, 1969). Water distribution networks is an important component of any water supply system accounting for up to 80% of the total cost of the system (Kleiner and Rajani, 2000) and as a result operation and maintenance cost may soar higher if they are poorly designed, hence the need to have a well planned, designed and constructed water distribution network cannot be over emphasized especially because of its importance to industrial growth and water's crucial role in society for health, fire fighting and quality of life (Taigbenu and Ilemobade, 2006).

A good water distribution network should meet the following requirements (Garg, 2000; Heavens and Gumbel, 2005):

(i) Water quality should not deteriorate in the system;
(ii) Every consumer should get sufficient water at desired pressure;
(iii) The design and layout should be economical;
(iv) The system should meet fire demand;
(v) It must afford easy maintainability; and
(vi) Deliver water of correct quality and quantity on a continuous basis with minimum service interruptions.

For a water distribution system to satisfactorily meet the above intended purposes, its hydraulics and design must be adequate and should be so constructed and operated that the chances for contamination of the water after it has entered the system are reduced to a minimum (Camp and Tawler, 1969). A hydraulic analysis of a water distribution network is required to determine the pressure contours and flow pattern of the system (Sincero and 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 (Quasim & others, 2000). This information enables the engineer to determine if the system is capable of meeting the demand for which it is being designed.

Direct solution of even simple networks as may be found in small communities is not possible (Sincero and Sincero, 1996), hence, several methods have been used to analyze distribution networks, the oldest and most widely used being 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. Such advances in water distribution network study include Computer Aided Water Supply Piping Network Design Model (WASDIM) presented in Nwaogazie and Okoye (1994).

WASDIM is a computer based model which utilizes the principles of Hardy Cross technique of network analysis which is a trial and error method by which corrections are applied to assumed piezometric heads at junction points until acceptable hydraulic balance of the system is achieved. The model permits accurate computation of rates of flow through the system and the resulting head losses by utilizing Darcy – Weisbach formula for head loss computation and Von- Karma's friction factor formula for turbulent flow condition. The software is written in FORTRAN 77 language and has capability to model grid, branched and combined system of grid and branched distribution network which typically characterize real life water distribution system.

The use of a hydraulic network model for water distribution planning and design is helpful to the engineer in the following respects (Ekenta,

(ii) Accuracy of computations is assured; and

(iii) The ease and speed with which models can be used gives the engineer the ability to explore more alternatives under a wide range of conditions resulting in more cost effective and robust design.

In this paper, we focus on the construction of a hydraulic network model (WaterCAD) and demonstrate its application in new development design of a small community water distribution network (Sakwa) in North Eastern geopolitical region of Nigeria.

2.0 HYDRAULIC ANALYSIS AND DESIGN OF WATER DISTRIBUTION NETWORK

The computation of flows and pressures in networks of pipes has been of great value and interest for those involved with the design, construction and maintenance of public water distribution systems and with the advent of computer models of water distribution systems, it has now become possible to analyze more complex network components (e.g. pumps, tanks, etc) of the water distribution system as well as to investigate more complex issues associated with their design and operation(Ormsbell,2006)

In a water distribution system, the steady state analysis is an important component of assessing the adequacy of a network. The hydraulic problem in connection with pipe networks consist of solving for the distribution of flow and head loss in the individual elements for a given total discharge or for a given total head loss. The steady state problem is considered solved when the flow pattern in each pipe is determined under some specified pattern of supply and consumption. The supply may be from reservoirs, storage tanks and / or pumps or specified as in flow or outflows at some points in the network and from the known flow rates the pressures or head losses through the system is computed. Alternatively, the solution may be initially for the heads at each junction or node of the network and these can be used to compute the flow rates in each pipe of the network.

or nodes and conservation of energy for each loop in the network. The continuity equation maybe expressed as

$$\sum Q_{in} - \sum Q_{out} = C_j \quad \text{------------------------------} \quad (1)$$

where Q_{in} and Q_{out} are the flow rates into and out of the junctions, respectively, and C_j represents external consumption or input flow rates at the junction

The energy principle provides that the head loss between any two points in the system is the algebraic sum of the head loss of all the elements along any route between the points and the total head loss is the same by all routes. The energy or loop equations are of the form

$$\sum h_{ij} = 0 \quad \text{------------------------------------} \quad (2)$$

for each loop.

Additionally, the head loss may be expressed by the power equation given as;

$h_f = K\, Q^n$ -- (3),

The values of K and n depend on the friction head loss equation adopted.

A pipe network of J – Junctions and L non overlapping loops and N – pipes will satisfy the equation (Jepson, 1976);

N = (J - 1) + L -- (4)

The number of independent equations which can be obtained for a network as described above is (J - 1) + L, Consequently, the number of independent equations is equal in number to the unknown flow rates in the N pipes. The (J - 1) continuity equations are linear and the L energy equations are non linear.

To solve for the unknowns, the equations must be solved simultaneously but the direct solution of the large number of simultaneous equations involved in distribution systems is impracticable for all but the simplest systems. Hence systematic methods which utilize computers are needed for solving this system of simultaneous equations.

WaterCAD is a windows-based software tool designed, developed and programmed by Haestad Methods Inc. of Cincinnati, Ohio, USA primarily for use in the modelling and analysis of water distribution systems.WaterCAD utilizes the gradient algorithm presented by Todini and Pilati modern day computer. In the formulation, individual energy equations for each pipe are combined with individual nodal equations for each junction node to provide for a simultaneous solution for both nodal heads and individual pipe flows (Ormsbee, 2006). The method can directly solve both looped and branched networks; is numerically stable when the system becomes disconnected by check valves, pressure regulating valves, or modeller's error and the structure of the generated equations allows for the use of extremely fast and reliable sparse matrix solvers (Haestad Methods, 2003).

2.2 PRINCIPLES OF NEW DEVELOPMENT DESIGN OF DISTRIBUTION NETWORK

In the design of a water distribution network for new developments using a network model, the following general principles or guidelines are adopted:

(i) Preliminary layout of all the proposed pipes is prepared on a suitable map of the community. Pipe sizes are assumed in accordance with underwriter's requirements or the designer's judgment (Brater and King, 1976). These sizes are checked or corrected by means of hydraulic computations and economic analysis that follow:

(ii) Initial pipe sizes can be set at either the minimum allowable size or a size estimated as suggested in Walski & others (2003) using the equation;

$$D = \sqrt{\frac{C_f Q}{V}} \quad \text{------------------------------} \quad (4)$$

where C_f = 1274 for Q in l/s, D in mm, V in m/s and V = maximum allowable velocity; and

(iii) The model is run for a variety of water demand loading conditions at the node (average day demand, maximum day demand, peak hour demand, maximum day demand plus fire flow (mdd + FF) at key locations and results reviewed for key issues as: (a) high velocities, (b) pressure below minimum, (c) low velocities during peak demand, (d) unusually high pressures, and (e) pumps not operating at desirable points on even during peak demand conditions and decreasing pipes sizes to determine the potential for cost savings without compromising velocity standards.

3. APPLICATION OF HYDRAULIC NETWORK MODELLING TO NEW DEVELOPMENT DESIGN

3.1 THE STUDY AREA

The study area, Sakwa is a small but rural community located between 12^0 16N latitude and longitude 10^0 21E in North Eastern geo-political region of Nigeria. Sakwa is within the Guinea Savannah zone with annual rainfall ranging from 400mm to 1100mm. The average mean monthly temperature varies from 15^0C to 35^0C. In this study it is required to design a distribution network to provide water for a future population of 4000 persons for the community using water demand of 60 litres per person per day in line with the standards prescribed by the Nigerian National Water Supply Policy as depicted in Table 1.

Table 1: Criteria Relating Population to Water Demand (NWSP, 2000)

Category	Population	Water demand
Urban	> 20,000	120 l/c/d
Semi-Urban	5000 – 20,000	90 l/c/d
Rural (Small)	< 5000	60l/c/d

3.2 WaterCAD MODEL VERIFICATION

Before utilizing the WaterCAD simulator for this study, the results obtained using the simulator was verified by applying the software to a simple water supply network with known solution (Jepson, 1976) and comparing the results obtained. The problem description (in imperial units) is presented in Figure 1; it is a pipe network fed from a tower 100ft above datum level. The network is composed of pipes with Hazen Williams pipe carrying capacity factor C= 120. The pipe lengths and their diameters are given in Table 2, and the preparation of input data for the WaterCAD computer simulation is made using the data in Figure 1and Tables 2 and 3. The head at junction 1 (H_1) is known and equal to 100ft and heads, H_2 and H_3 at junctions 2 and 3 are H_2 = 91.45ft and H_3=90.84ft. The results obtained for the flows in the pipes of the simple network (Jepson, 1976) solved with Newton Raphson method of Network analysis (Q_{12}=1103.35 gpm, Q_{13}=918.24 gpm, and Q_{23}= 428.15gpm) were compared with that obtained by application of WaterCAD simulator to the same network and is presented in Table 4.

Table 2: pipe input data for WaterCAD simulator

Pipe label	Length (ft)	Diameter(in)	C
P-0	1	36	120
P-1	1000	10	120
P-2	1000	12	120
P-3	1500	10	120

Table 3: Nodal input data for WaterCAD simulator

Node label	Elevation(ft)	Demand (gpm)
1	100	0
2	91.45	673.2
3	90.84	1346.46

Table 4: Comparison of values of discharge along pipes

Pipe label	Discharge label	Jepson solution (gpm)	WaterCAD solution (gpm)	%Difference in discharge values
P-1	Q_{12}	1101.35	1101.26	0.008
P-2	Q_{23}	428.15	428.06	0.035
P-3	Q_{13}	918.24	918.34	0.01

Comparisons of Jepson's solution and WaterCAD simulator values for flow along the pipes are presented in Table 4. The percent difference in the flow rate values ranges from 0.008% to 0.035% and hence WaterCAD simulator results are considered reliable and adequate and thus acceptable for this study.

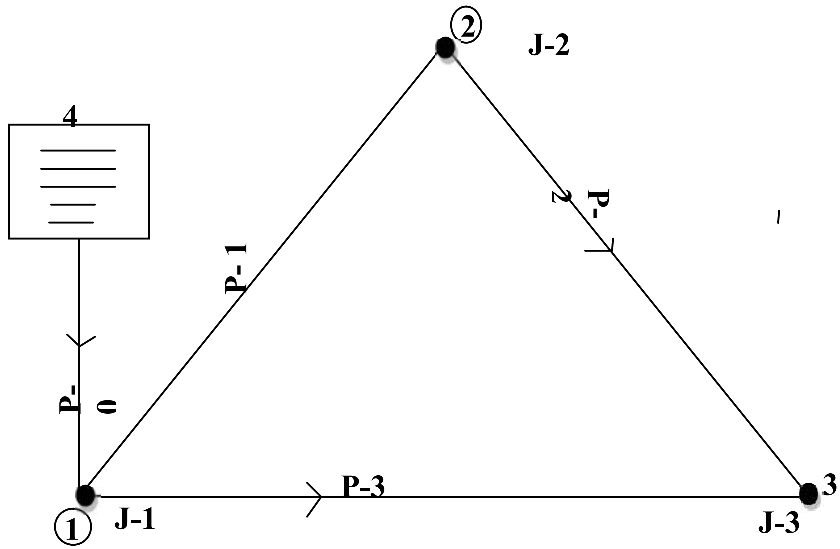

Figure 1: Idealized water distribution network (Jepson, 1976)

3.3 DEVELOPMENT OF DISTRIBUTION SYSTEM NETWORK MODEL FOR COMPUTER SIMULATION

3.3.1 Existing Components of the System

The existing components of the water supply scheme for which a distribution network is to be designed are:

(1) 2 Nos 150mm diameter boreholes with safe yields of 6 l/s and 7 l/s and connected to the existing over head tank

(2) A 45m^3 steel tank elevated on a 10 metre high tower

3.3.2 Construction of the Network Model of Sakwa Community

A distribution system is designed to deliver peak hourly demand to each customer (Al-Layla and others, 1978). The distribution pipes were proposed to follow the right of way of the main streets in the community in order to provide pipeline access to every potential user (Qasim and Others, 2000).

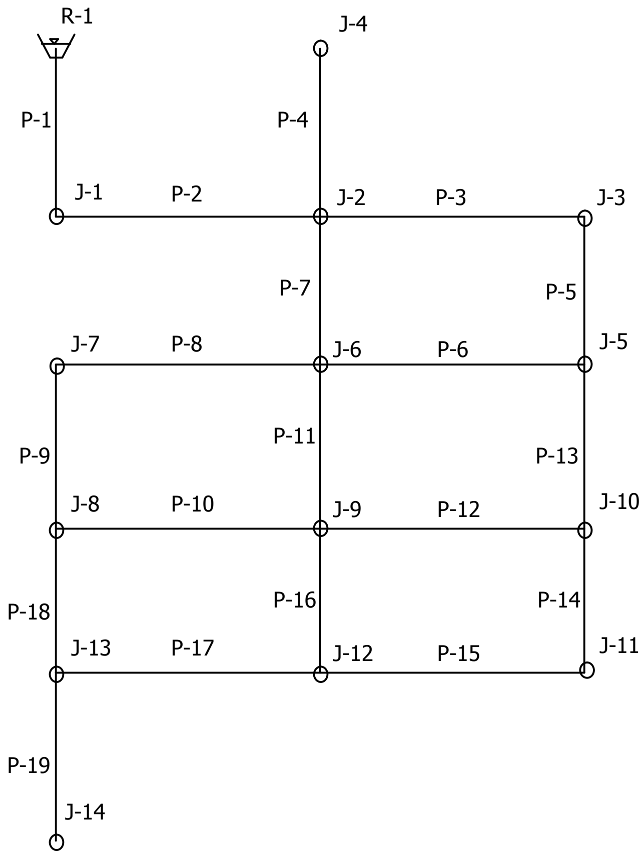

Figure 2: WaterCAD model of Sakwa Community distribution network

The hydraulic model of the proposed Sakwa Community distribution network was constructed as a link-node connectivity of the various elements of the network using WaterCAD Version 6 software tool pallete as shown in Figure 2. The model consists of 19 pipe segments, 14 nodes and 1 reservoir.

3.3.3. Water demand loading or distribution

Water demand is the driving force behind the hydraulic dynamics occurring in water distribution system and water use is spatially distributed as demands or loads assigned to model nodes. The system water demands were distributed to model nodes or Junctions using the guidance of Griffiths (1980) as follows:

(i) The supply area is divided into such areas each commanded from the intersection of two principal mains in the

(ii) The estimated average daily demand (**Add**) at the end of the design period is calculated. Peaking factor of 1.5 was applied to **Add** to obtain maximum daily demand (**Mdd**) while a peaking factor of 1.5 was applied to **Mdd** to obtain peak hourly demand (Agunwamba, 2000)

(iii) The total demand along a main is split 50% to each end to obtain the demand at a node (Anyata, 1980; Twort, 1985)

(iv) Fire flow of 10l/s (Al-Layla and others, 1978) is added to maximum day flows of a node when evaluating the capacity of the system for fire fighting.

Using the above procedure the nodal demand input data for the study network (Fig. 2) was obtained as given in Table 5 while the corresponding output data is presented in Table 7.

Table 5: Nodal Demand Input Data

Node Label	Elevation	Add (l/s)^	Mdd (l/s)+	Pkhd (l/s)**	Mdd + FF*
J – 1	100	0.373	0.56	0.84	
J – 2	100	1.01	1.52	2.24	
J – 3	100	0.53	0.80	1.2	
J – 4	100	0.16	0.24	0.36	
J – 5	100	0.69	1.04	1.56	
J – 6	100	1.01	1.52	2.28	11.52
J – 7	100	0.48	0.72	1.08	
J – 8	100	0.59	0.89	1.34	
J – 9	100	0.91	1.37	2.06	11.37
J – 10	100	0.59	0.89	1.34	
J – 11	100	0.43	0.65	0.98	
J – 12	100	0.91	1.37	2.06	11.37
J – 13	100	0.43	0.65	0.98	
J – 14	100	0.16	0.24	0.36	

Key: **^**Add = Average daily demand; **+**Mdd=Maximum daily demand; **Pkhd= Peak hourly demand, *FF = Fire flow

3.4 DESIGN CRITERIA FOR STUDY

The criteria adopted for this study are service pressures, flow velocities and hydraulic loss gradients

3.4.1 Service pressure criteria

The pressures required in the mains for normal domestic consumption depends upon the height of the buildings served directly without pumping within the buildings, the maximum instantaneous rate of flow through the house service pipes and friction losses in meters. Also, it is necessary to maintain sufficient pressure in the distribution system in order to protect it against contaminants by ingress of polluted seepage water. For small community water supply systems as in the system being studied, a minimum pressure of 6 metre head of water is considered adequate in most instances (Hofkes and others, 1991) and it is adopted for this study.

3.4.2 Flow velocity criteria

No fixed criteria exists regarding the maximum velocity in a main (Walski and others, 2003) but pressures usually start to drop off and water hammer becomes more pronounced when velocities reach 3m/s. And to prevent sediments from accumulating in drinking water distribution systems it is recommended that networks be designed as self cleansing (Vreeburg and others, 2008). Though the threshold design velocity for self cleansing drinking water distribution system is set at 0.4m/s, this value is considered conservative hence, a regular occurring velocity of 0.2m/s or less may be enough (Vreeburg and others, 2008). Hence, for this study a maximum value of 1.5m/s and minimum value of 0.2 m/s were adopted for flow velocities in the distribution mains.

3.4.3 Hydraulic loss gradient criteria

Based on the American Water Works Association (AWWA) recommendations (Akdogen, 2005), the following criteria for head loss gradients in Table 6 were adopted for this study.

Table 6: AWWA Recommendations for Hydraulic Loss Gradients+

Diameter (mm)	J(m/km)
80	50
100	35
125	25
150	15.22
200	6.62
250	2.88
300	1.25
350	0.54

\+ **(Akdogen, 2005)**

3.5 STEADY STATE ANALYSIS OF NETWORK MODEL

The network model was analyzed for steady state (static) simulations. Four water demand loading conditions were applied at the nodes namely: average day demand (Add), maximum daily (Mdd), peak hour demand (Pkhd) and maximum daily demand plus fire flow (Mdd + FF) at critical locations within the network for alternative systems using principles stated in item 2.2 above. The corresponding output results were appraised and compared using the study design criteria from which the optimized network is obtained.

4. PRESENTATION AND DISCUSSION OF RESULTS

The nodal pressures obtained in the study network for three water demand levels of Add, Mdd and Pkhd with the existing overhead tank at 10 metres above ground level is given in Table 7. It is the output data obtained by applying the nodal input data at the three water demand levels (Table 5) to the study network (Fig.2).

Table 7: Nodal Pressures within Network with Tank Tower Height of 10m (Nodal Output data)

	Nodal Pressures (m)		
Node	**Add**	**Mdd**	**Pkhd**
J – 1	9.91	9.83	9.66
J – 2	9.52	9.00	7.90
J – 3	9.47	8.90	7.70
J – 4	9.52	9.00	7.89
J – 5	8.91	7.72	5.19
J – 6	8.91	7.71	5.18
J – 7	8.71	7.30	4.30
J – 8	8.69	7.25	4.20
J – 9	8.75	7.37	4.45
J – 10	8.75	7.37	4.45
J – 11	8.72	7.32	4.34
J – 12	8.72	7.31	4.32
J – 13	8.69	7.24	4.17
J – 14	8.68	7.23	4.15

At overhead tank height of 10 metres (see Table 7), the system would be unable to deliver peak hourly demand at ten of the fourteen nodes within the network at the minimum of 6m pressure

(Fig.2) by applying the nodal demand input data (Table 5) but with the existing tank tower height increased from 10 to 15 metres for four water demand levels, that is, Add, Mdd, Pkhd, and for of 4.81m is recorded at node J – 14. Considering the height of buildings in this small community this is considered acceptable, hence more satisfactory service will be obtained by increasing the existing overhead tank height to 15 metres.

Table 8: Nodal Pressures within Network with Tank Tower Height of 15m (Nodal output data)

Node	Nodal Pressure (m)				
	Add	**Mdd**	**Pkhd**	**Mdd + FF_6**	**Mdd + FF_9**
J – 1	14.90	14.82	14.65	14.52	14.52
J – 2	14.51	13.99	12.89	11.94	11.94
J – 3	14.46	13.89	12.69	11.67	11.64
J – 4	14.50	13.98	12.88	11.93	11.93
J – 5	13.90	12.71	10.18	7.53	7.11
J – 6	13.90	12.70	10.17	6.76	6.96
J – 7	13.70	12.29	9.29	6.43	5.33
J – 8	13.68	12.24	9.19	6.41	4.87
J – 9	13.74	12.36	9.44	6.55	4.83
J – 10	13.7	12.36	9.44	6.69	5.36
J – 11	13.71	12.31	9.33	6.58	5.13
J – 12	13.71	12.30	9.31	6.51	4.82
J – 13	13.68	12.23	9.16	6.40	4.82
J – 14	13.67	12.22	9.14	6.39	4.81

Table 9 presents the comparison of the summarized pipe reports for five water demand conditions simulated in the analysis when the elevated tank is of 15 metre height. The head loss gradients (Hlg) and flow velocities (V) along the pipe lines for each of the five water demand conditions are given in the table and from which it can be seen that the values of the head loss gradients along all the pipelines fall within acceptable values recommended by American Water Works Association (AWWA) for the different pipe sizes as given in Table 6. Also, Table 9 shows that low flow velocities exists along pipes P-4, P-6, P-12, P–15 and P – 19 even at peak demand conditions, suggesting that the pipe diameters be decreased below the selected sizes for cost savings. However, this is not practical because pipes: P – 6, P – 12, P – 15 and P – 19 are of 75mm diameter which is the minimum commercially available pipe size for uPVC or AC pipes that are predominantly used for distribution mains in Nigeria.

Table 9: Summary of Pipe Reports for Various Simulations for elevated tank height of 15m

Pipe Label	Pipe Diameter mm)	Pipe length	Add		Mdd		Pkhd		mdd + FF_6		mdd + FF_9	
			Hlg (m/km)	V (m/s)	Hlg (m/km)	V (m/s)	Hlg (m/km)	V (m/s)	Hgl (m/km)	V (m/s)	Hgl (m/km)	V (m/s)
P – 1	150	50	1.43	0.47	3.03	0.70	6.41	1.05	9.04	1.3	9.04	1.3
P – 2	150	300	1.31	0.45	2.78	0.67	5.89	1.06	8.63	1.2	8.63	1.2
P – 3	150	300	0.15	0.14	0.32	0.21	0.67	0.31	0.9	0.4	0.98	0.4
P – 4	100	150	0.01	0.02	0.01	0.03	0.03	0.05	0.01	0.03	0.01	0.03
P – 5	75	200	2.80	0.44	5.91	0.65	12.57	0.98	20.73	1.3	22.7	1.35
P – 6	75	300	0.01	0.02	0.01	0.02	0.03	0.04	2.58	0.4	0.51	0.11
P – 7	100	200	3.03	0.55	6.43	0.82	13.62	1.23	25.9	1.74	24.9	1.69
P – 8	75	300	0.55	0.20	1.39	0.30	2.94	0.45	1.07	0.3	5.42	0.62
P – 9	75	150	0.15	0.09	0.32	0.14	0.68	0.20	0.17	0.09	3.09	0.46
P – 10	75	300	0.18	0.10	0.39	0.15	0.82	0.23	0.46	0.16	0.14	0.09
P – 11	100	150	1.09	0.31	2.31	0.47	4.9	0.71	1.4	0.36	14.22	1.3
P – 12	75	300	0.00	0.01	0.01	0.12	0.01	0.03	0.48	0.17	1.79	0.34
P – 13	75	150	1.10	0.27	2.33	0.04	4.93	0.59	5.6	0.63	11.65	0.94
P – 14	75	100	0.25	0.12	0.53	0.18	1.12	0.27	1.11	0.26	2.37	0.4
P – 15	75	300	0.01	0.02	0.02	0.03	0.04	0.05	0.25	0.12	1.01	0.25
P – 16	100	100	0.27	0.15	0.57	0.22	1.22	0.33	0.42	0.19	0.03	0.05
P – 17	75	300	0.11	0.08	0.02	0.120	0.51	0.17	0.34	0.14	0.02	0.03
P – 18	75	100	0.06	0.06	0.13	0.08	0.28	0.13	0.07	0.06	0.5	0.17
P - 19	75	150	0.03	0.04	0.06	0.05	0.13	0.08	0.06	0.05	0.06	0.05

Though flow velocity along P – 4 is low, the 100mm size is considered adequate because it will permit some moderate further extension (Twort and others, 1986) as demand is expected to increase on the pipe segment added to the fact that under normal operating condition maximum velocities in distribution mains can sometimes be very low (< 0.01m/s) (Vreburg and others, 2008) because the distribution network is usually designed for fire flow demands that are typically much higher than domestic demands.

5. CONCLUSIONS AND RECOMMENDATIONS

The application of WaterCAD simulator for hydraulic analysis and design of a small community water distribution network is presented. The following conclusions are based on the presentations:

(i) Computation of flows and pressures in network of pipes is of great value and interest for those involved with design, construction and maintenance of public water distribution systems;

(ii) Computer aided modelling of water distribution network offers advantages over manual computations as it enables a system to be modelled relatively quickly, thereby saving time from repetitive iterations that determine the flows and pressures;

(iii) Network models facilitate hydraulic analysis and design of a water distribution network and such models as EPANET, WaterCAD, and WASDIM have been used successfully by research and applications communities alike;

(iv) Network models play a vital role in the design, operation and management of water distribution system today and in impacting the quality of life in our communities;

(v) Though computer aided modelling software saves the engineer the time in performing repetitive iterations to solve pipe network system, the engineer still needs to understand hydraulic engineering principles in order to make sound assumptions, accurately input data fields and understand model outputs;

(vii) The selected pipe sizes for the Sakwa water distribution network is adequate and will meet peak water demands and fire protection while maintaining adequate or prescribed pressure in the system;

(viii) The existing tank elevated on a 10 metre high tower will not provide satisfactory pressures within the network;

(v) Increasing the tower height of the existing tank to 15m and utilizing diameters selected for the network will yield adequate and economic design ;

(vi) Flow velocities within the system will not be excessive; and

(vii) Head loss gradients in the distribution network are within acceptable limits

Arising from the above conclusions the following recommendations are made:

(1) The existing elevated tank tower be increased from 10m to 15m

(2) The materials of make for the pipe should be either PVC or uPVC which are environmentally friendly and predominantly used in this country ; and

(3) Where the design of self cleansing networks is not readily realized as with small community water distribution network, it is recommended that sediment be prevented from entering the distribution system through optimizing water treatment and by removal of sediments by flushing the system in a timely manner

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