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Abstract
This study presents a method for optimizing the design of a water distribution network using pipe diameter as a decision variable under the required demand loading and hydraulic conditions. Data was collected from field studies and an optimization model was formulated from the obtained information using pipe diameter as decision variable. Hydraulic conditions were simulated with EPANET 2.0 software while the optimization model was solved using LINGO 13.0 software. The result of the study revealed that an increase in minimum pressure will lead to reduction in the required pipe diameter. Also the predominant pipe sizes in the optimum solution of the model were 100 mm and 150 mm. However, at higher values of minimum pressure, pipes of larger diameters were not required to obtain the optimal solution to the water distribution system. The optimization model developed will be very useful for determination of economical pipe sizes in a water supply system for newly planned layout and for evaluating existing distribution system for upgrading and strengthening.

Keywords: *Optimization, Model, Water Distribution, Network, Design*.

1.0 Introduction
A water distribution system connects consumers to sources of water, through hydraulic components, such as pipes, valves, and reservoirs. Once pipelines are in place, land use planners rightly press for the lowest cost of expansion, which is along the pipeline route. As a result, even though these utilities initially respond to growth, the latter are impetus for urban and rural expansion (Ashton and Bayer 1983). A distribution network is an essential part of all water supply systems and its cost, in any sizable water supply scheme, amounts to more than 60% of the entire cost of the project (Abdel-Gawad, 2001). Also, the energy consumed in a distribution network is more than 80% of the total energy consumption of the system. The high investment and maintenance costs associated with both new water distribution networks and the expansion of existing ones have led hydraulic researchers to take great interest in mathematical methods to find their optimal design, that is, the minimum cost network. Over the last three decades a large number of optimization models have been developed aiming at minimizing the cost incured on distribution systems (Abdel-Gawad, 2001).

Al-Zahrani and Syed (2004, 2005) developed a methodology for evaluation of water system reliability using minimum cut-set method. It was suggested in the work that appropriate number of pipe closure combinations should be selected based on field experience in order to get realistic values of nodal and system reliability. Vamvakeridou-Lyroudia, et al. (2007) proposed storage optimization technique aiming to bridge the gap between traditional engineering practice and mathematical considerations needed for genetic algorithms (GAs). The major variable used for the optimization was only limited to tank simulation whereas there are other essential variables and components that constitute water distribution systems. Sumer and Lansey (2009) studied the effect of uncertainty on water distribution system model design decisions. However, this work did not incorporate the entire components of water distribution network systems. Kumar, et al. (2008) described a state estimation technique for well instrumented water distribution networks graph-theoretic.

The work looked beyond the individual network solution strategies currently available that are meant for obtaining flows/pressure given a minimum number of measurements. Filion (2009) examined the challenge that utilities face in designing new transmission systems with incomplete information concerning future water demand. He therefore presented an analytical probabilistic model that evaluated the expected level and uncertainty of pressure head at the end of design period linked to pipe-diameter decisions in a new water transmission pipeline subject to uncertainty in water demand. However, the approach is only suitable for simple systems where the relationship between demand and pressure head is resolved through a single closed-form equation.

The operational control of water networks has posed difficulties in the past to the human operator that had to take the right decisions, such as pumping more water or closing a valve, within a short period of time and quite frequently in the absence of reliable measurement information such as pressure and flow value. This is because the water networks are large scale and non-linear systems. To tackle these challenging difficulties, Arsene, et al. (2005) worked on modeling and simulation scheme for water distribution systems based on loop equations. The purpose of the work was to investigate the implications of the loop equations formulation of simulator algorithm, state estimation procedure and confidence limit analysis for the implementation of decision support systems in operational control of water networks. Coulbeck and Orr (1984) opined that water distribution networks are generally the least well defined of the three major utilities (i.e. gas, water and electricity). There are a number of reasons for this which probably includes: measurement difficulties, high cost of telemetry, complex and interactive networks, empirical network relationships, and undetected leakage.

Tanyimboh and Sheahan (2002) investigated the idea that minimum cost maximum entropy designs of water distribution systems can be used to identify good layouts of water distribution systems in the sense that designs based on these layouts could achieve a reasonable compromise between reliability and cost. However, it was concluded that more research is required, notably in respect of the extension of the method to more realistic systems and the handling of multiple demand patterns. Kalungi and Tanyimboh (2003) proposed a solution procedure to the pressure head driven model based on a globally convergent Newton algorithm. The pressures at partial flow nodes and key partial flow nodes are corrected at every iteration, using the globally convergent Newton algorithm till the nodal pressures at all the partial flow nodes fall below the desirable head. Kessler, et al. (1990) stated that water distribution networks, like other engineering systems, are subjected to random failures which may occur in the elements of the network (mostly pipes) or in the nodes (pipe junctions and demand nodes). In their conclusion, they suggested that further research is needed in order to determine the relation between the pressure redundancy and the desired level of service under emergency conditions.

Developing methodologies for the minimum cost design of water supply networks, optimizing coverage with satisfaction has been under investigation for the last four decades and with the advent of computers a radical change is taking place in the planning and design of water distribution pipe network (Letha and Sheeja, 2003). Liong and Atiquzzaman (2004) used EPANET, a widely used water distribution network simulation model, to deal with both the steady state and extended period simulation which were linked with a powerful optimization algorithm, Shuffled Complex Evolution (SCE). It has been shown that SCE is a potential alternative optimization algorithm to solve water distribution network problems. Awumah, et al. (1990) presented entropy based expressions for measurement of reliability and redundancy of distribution network. Other articles have been published by many other researchers such as Eiger, et al. (1994), Alperovits and Shamir (1977), Collins, et al. (1978), Abel-Gawad (2001), Biscos, et al. (2003), Liong and Atiquzzaman (2004), Perelman and Ostfeld (2007), Afshar and Jabbari (2008), Sarbu (2009), Cisty and Bajtek (2009), Vasan and Simonovic (2010) and lot more. The aim of all these articles was to find optimal design of water distribution that minimized the system cost. This paper used Non Linear Programming to formulate optimization model using diameter as a decision variable under various hydraulic loading. The model was solved using LINGO software by considering a case study applicable to Nigeria as a developing nation. The work will serve as a guide for the stakeholders in taking decision related to water distribution network design for minimum cost at required minimum residual pressure.

2.0 Materials and Method

The methodology of this study comprises of field data collection and analysis. Field work was conducted in order to determine some of the parameters needed for the optimization model (such as link diameter, length, and nodal elevation and all other fittings and appurtenances that may be incorporated in the network). Pipe network analysis seeks to determine the discharge and pressure at every node. In order to carry out this, the physical features of the network must be known. These features are link diameter, length, nodal elevation and all other fittings and appurtenances which can be shown on the network layout. The hydraulic Model EPANET 2.0 (Rossman, 2000) was used to perform hydraulic simulation of the water distribution system as earlier explained. Several commercial simulation models are available, but in this study EPANET (2000) was used to perform hydraulic simulation. It was selected because it fulfilled the requirement of calculating nodal pressures, and also its source code is available free of cost in the public domain, and can be applied to large water distribution networks with unlimited pipe numbers. EPANET uses the same numerical engines as WaterCAD and other commercially available software. The optimization model formulated was based on link diameter as a decision variable and used the link cost function as suggested by Bhave (2003). The optimization model was solved using LINGO 13.0 software. The LINGO software is a comprehensive tool designed to make the building and solving of mathematical optimization models easier and more efficient. LINGO provides a completely integrated package that includes a powerful language for expressing optimization models, a full-featured environment for building and editing problems, and a set of fast built-in solvers capable of efficiently solving most classes of optimization problems.

The study area considered for this study was Irewode Housing Estate in Ilorin, Kwara State, Nigeria, a residential estate comprising of several units of family size buildings (Figure 1). The skeletonized network layout for the study area is shown in Figure 2 while the data obtained from the field work is shown in Tables 1 to 3.

Table 1: Case study area parameters

Parameters	Unit	Values
Average houshold Population	No	6
Total Numbers of Building	No	250
Total Numbers of Households (2 x 250)	No	500
Estimated Total Population (500 x 6)	No	3,000
Average Water demand	l/c/d	86.22

(Source: Ayanshola 2013)

Estimation of Total water demand for the area was determined as follow:

Average Standand Water Demand (WHO) = 120 l/c/d (This value is adopted for planning and design since it is greater than the estimated value in Table 1)

Total water demand = 120 x 3,000 = 360,000 l/d

Using a peak hour factor of 2.0 = 360,000 x 2 l/d = 8.33 l/s

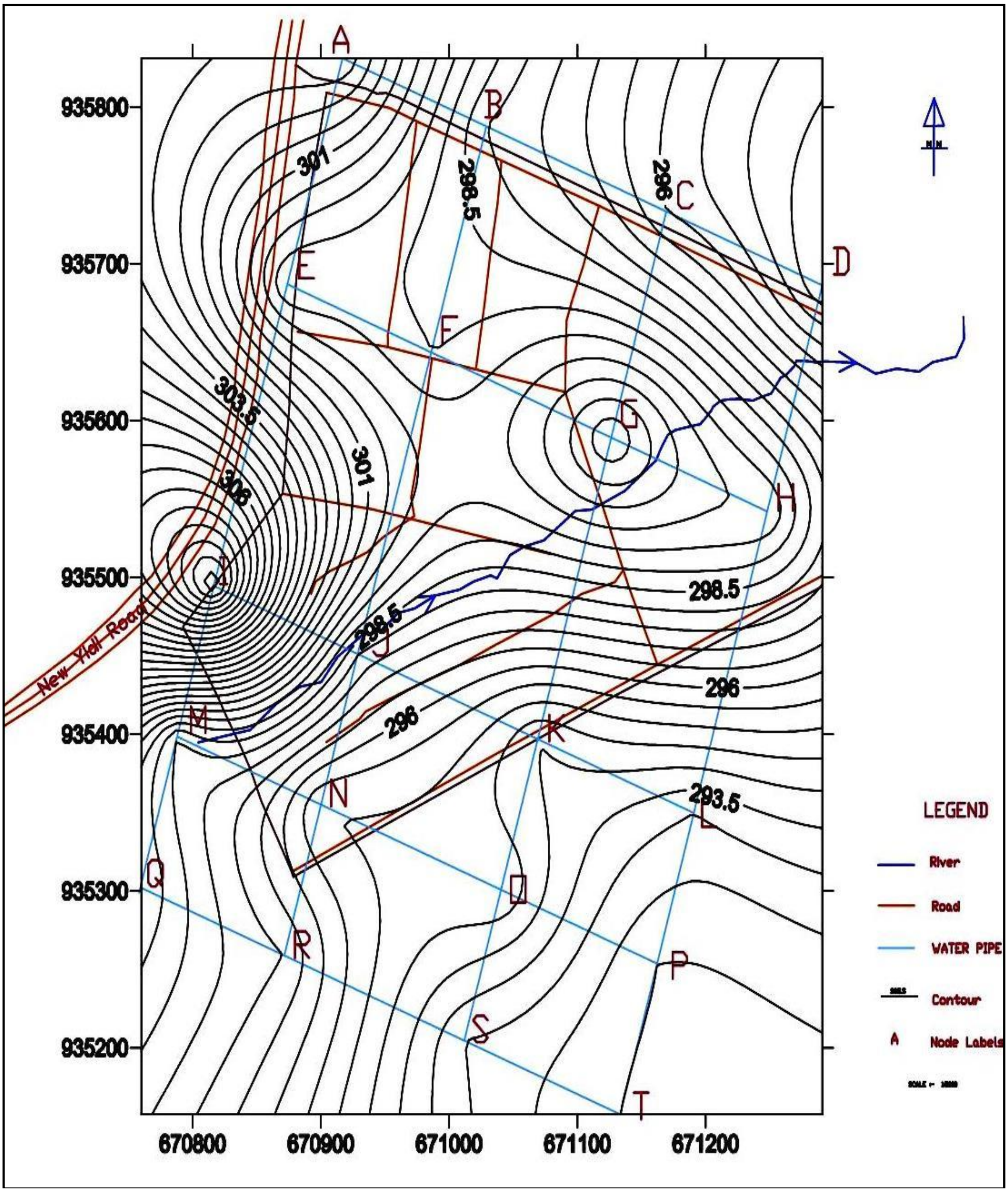

Figure 1: Layout and Contour Map of the Study Area

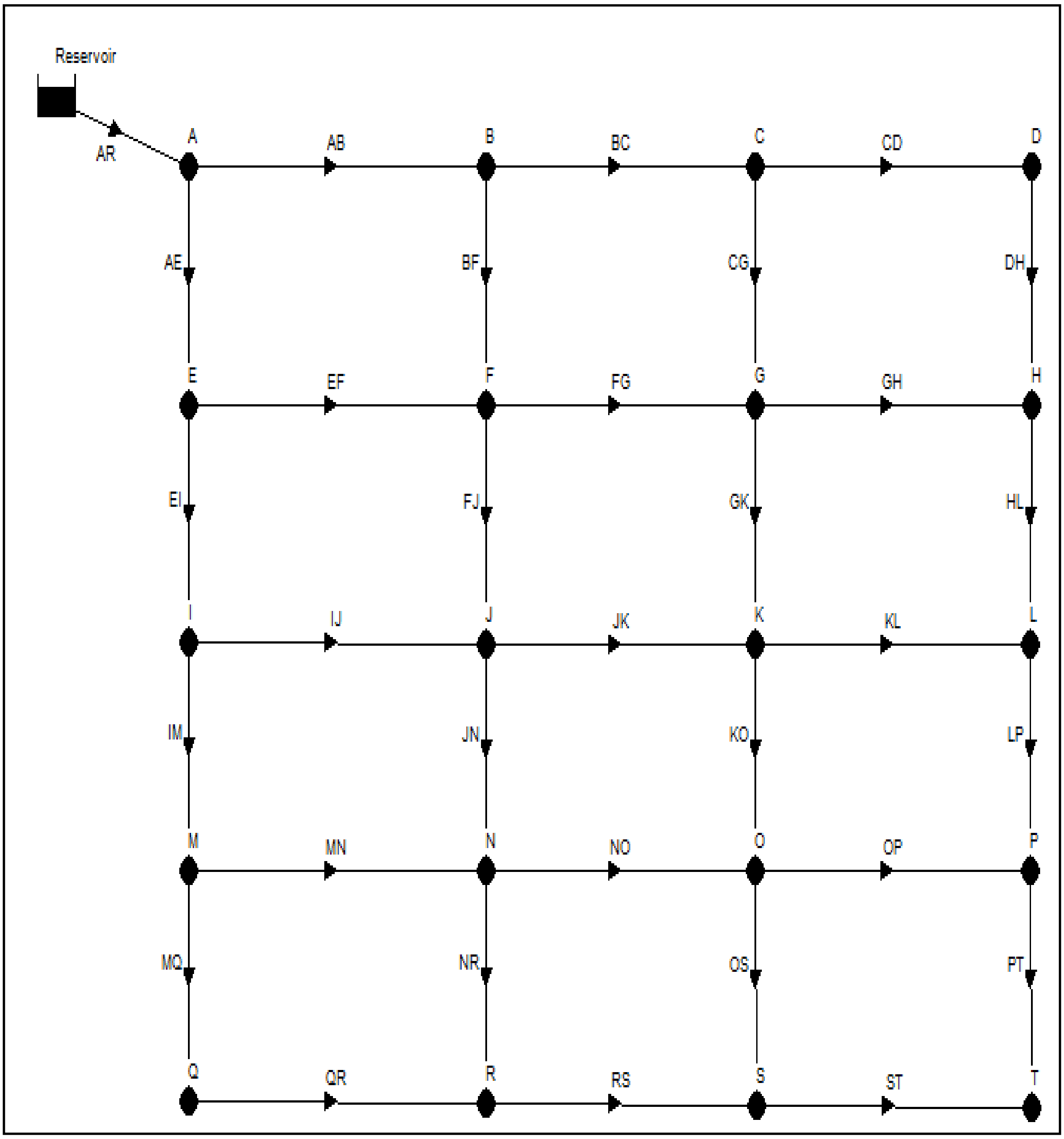

Figure 2: Skeletonized Pipe Network Layout for Irewolede Housing Estate, Ilorin, Nigeria

Table 2: Link parameters and estimated average flow through them

Links	Length of Links (m)	Diameter of links (mm)	Flows along the links, Q (l/s)	Q/2
AB	120	250	0.238	0.119
BC	150	150	0.298	0.149
CD	130	100	0.258	0.129
DH	150	150	0.298	0.149
HG	130	100	0.258	0.129
GC	150	200	0.298	0.149
GF	150	150	0.298	0.149
FB	150	100	0.298	0.149
FE	120	250	0.238	0.119
EA	150	100	0.298	0.149
EI	200	200	0.397	0.198
IJ	120	150	0.238	0.119

JF	200	200	0.397	0.198
JK	150	100	0.298	0.149
KG	200	250	0.397	0.198
KL	130	100	0.258	0.129
LH	200	250	0.397	0.198
LP	100	150	0.198	0.099
PO	130	200	0.258	0.129
OK	100	100	0.198	0.099
ON	150	150	0.298	0.149
NJ	100	200	0.198	0.099
NM	120	100	0.238	0.119
MI	100	150	0.198	0.099
MQ	100	150	0.198	0.099
QR	120	100	0.238	0.119
RN	100	200	0.198	0.099
RS	150	250	0.298	0.149
SO	100	150	0.198	0.099
ST	130	100	0.258	0.129
TP	100	200	0.198	0.099
TOTAL	4200		8.333	

3.0 Formulation of the model

3.1 Cost Function

The costs of transmission mains and distribution network are primarily due to the costs of pipes (Bhave, 2003). For this work, unplasticized Polyvinyl Chloride (UPVC) with 16 bar pressure rating has been selected being a common pipe material mostly used and widely available. The market price for the pipe material was therefore used to generate cost function as shown in Figure 3. Hence, the generated cost of installation C of a link is expressed as:

$$C = BL^{a}D^{m} \tag{1}$$

where B = cost coefficient; L = link length (m); D = link diameter (mm); a, m = exponents obtained by regression analysis.

It is however common to assume that the cost of a link varies linearly with length (a = 1) so that equation (1) reduces to:

$$C = BLD^{m} \tag{2}$$

Consequently, the cost function for UPVC pipe cost data shown in Figure 1 is given as:

$$C = 0.254 L_x D_x^{1.834} \tag{3}$$

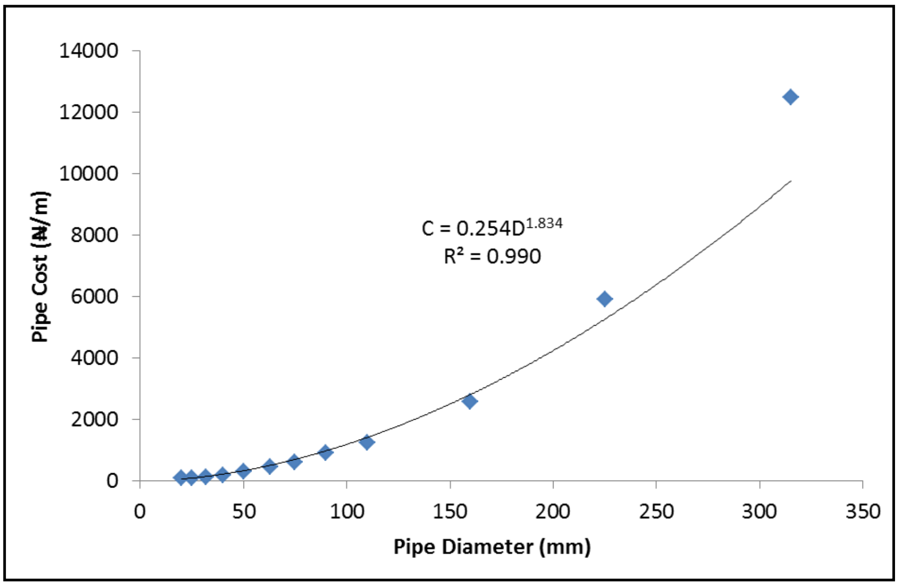

Figure 3: Pipe Diameter – Cost Relationship for UPVC 16 bar Pressure Pipe

3.2 Optimization Model

3.2.1 Objective Function

The objective function is minimizing the total cost of installation, C_T; i.e.

$$Min\ C_T = \sum_{x=1}^{X} 0.254 L_x D_x^{1.834} \quad (4)$$

where L_x is the length of link x in m and D_x is the diameter of link x in mm

3.2.2 Constraints

There are four constraints to be satisfied by the objective function

a) Node flow continuity Constraints

$$\sum_{x\,incident\,on\,j} Q_x + q_j = 0, \qquad j = 1, \ldots, N \quad (5)$$

where Q_x is the flow in link x to the node j; q_j is the demand at node j and; N is the total number of nodes in the network.

b) Loop head constraints

$$\sum_{x \in c} A L_x Q_x^p D_x^{-r} = 0, \qquad c = 1, \ldots, C \quad (6)$$

where A is the constant depending on link material, and units of different terms; P and r are exponents equal 2 and 5 respectively for Darcy-Weisbach formula; 1.85 and 4.87 respectively in Hazen-William formula.

$$A = \frac{\alpha}{C_{HW}^{1.85}} \quad (7)$$

where $\alpha = 2.234 \times 10^{12}$ (Bhave, 2003) and; $C_{HW} = 130$ for UPVC, hence, $A = \frac{\alpha}{C_{HW}^{1.85}} = 2.74 \times 10^{8}$

c) Path head loss constraints

$$\sum_{x \in pj} \boldsymbol{A L_x Q_x^p D_x^{-r}} \leq \boldsymbol{h_j^{\min}}, \qquad j = 1, \ldots, N \tag{8}$$

Where: $\boldsymbol{h_j^{\min}}$ is the minimum residual pressure allowed at node *j*

d) Non-negativity constraints

$$D_x \geq D^{min},\ Q_x \geq Q^{min} \tag{9}$$

where; D^{min} *and* Q^{min} are respectively the minimum link diameter and flow allowed in the network.

In solving the formulated optimization model for the water distribution network and to estimate other associated parameters, a software (LINGO 13.0 Version) was used. The model formulated had 64 variables and 129 constraints (45 non-linear). For all the analysis performed, the number of iterations ranged between 260 and 421. The minimum pipe diameter considered was 75 mm and the minimum residual pressure was varied so that the effect of the pressure can been seen on the optimum cost.

4.0 Results and Discussion

4.1 Objective Values

Figure 2 shows that optimum cost is inversely proportional to the pressure. Increase in minimum pressure resulted in the reduction of the installation cost.

4.2 Pipe Diameters

Since the size of pipe also determine the cost of installation, it has been shown that as the residual pressure is increased, the required pipe diameter will be reduced. Figure 4 shows the number of pipes within the ranges of pipe diameter at various minimum residual pressure. It can be seen that with the highest minimum pressure considered (30 m), the smallest pipe size, i.e. 75 mm is predominant which will definitely generate the minimum cost. Hence, as the minimum pressure is increased, the the pipe diameter needed decreases to meet up with the loading condition of the network.

The number of pipes whose sizes are between 100 mm and 150 mm are the next highest to the smallest pipe size considered. This can be seen in Figure 5 more clearly where the curves peaks at (100 mm to 150 mm) diameter range at minimum pressure of 8 m and 10 m. The curves has indicated clearly that increase in pipe diameter result in the use of fewer number of pipes in the network. At higher values of minimum pressure, pipes of larger diameters were no longer required to obtained the the optimal solution to the water sdistribution system.

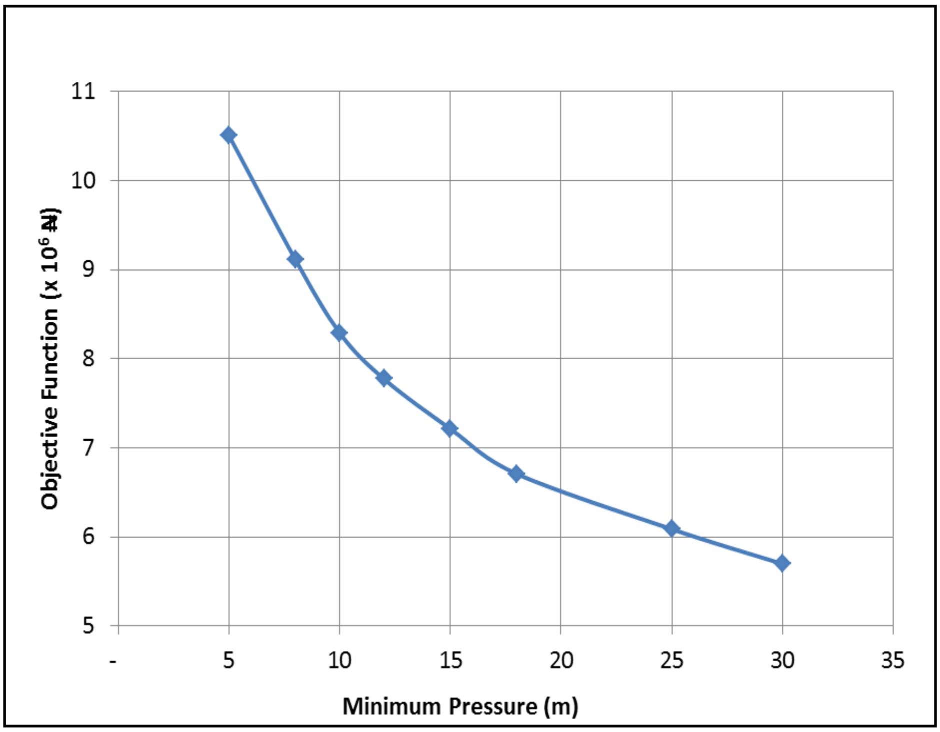

Figure 4: Optimal Objective Function Values

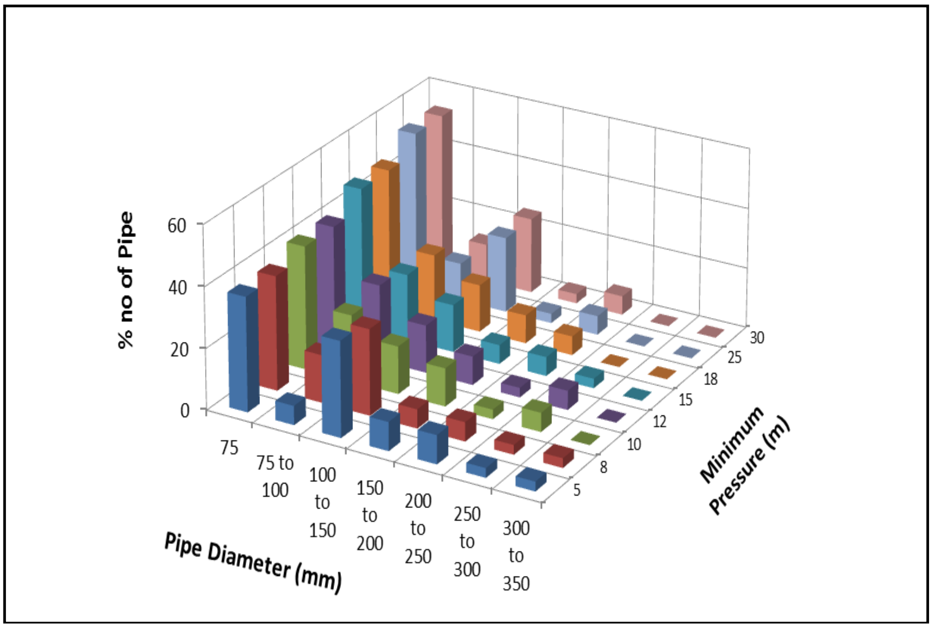

Figure 5: Percentage No of Pipes and Range of Pipe Diameter for Various Minimum Residual Pressures

4.3 Pipe Flows

In Table 4, pipe AR has the highest value of flow for all cases of minimum pressure considered. This is because the total volume of water that will serve the whole areas will flow through the pipe. Pipes KO and LP at 8 m pressure and pipe DH at 10 m pressure were found to be redundant because they had zero flow. Such redundant pipes will only be useful when there is problem along the other pipes serving the same area. Pipe RS had the minimum flow at 5 m, 12 m and 15 m pressure. At 18 m pressure, pipe HL had the minimum flow while at 25 m and 30 m pressure, pipe PT was found to have the minimum value of flow. Table 4 results imply that minimum pressure of 12 m is needed to avoid pipes with zero flows.

Figure 6 shows the number of pipes within various flow ranges at stipulated minimum pressure. The pipes flows flows less than 0.2 l/s were decreasing in number with increase in minimum pressure. But in the case of flow ranges between 0.2 l/s and 0.5 l/s, the numbers of pipe increased with increase in minimum pressure. The number of pipe within these ranges of flow (0.2 l/s and 0.5 l/s) were also predominant in the network as shown in Figure 7. At higher values of minimum pressure, pipes with high values of flow were very limited in number in the optimal solution of the network design case (Figure 8).

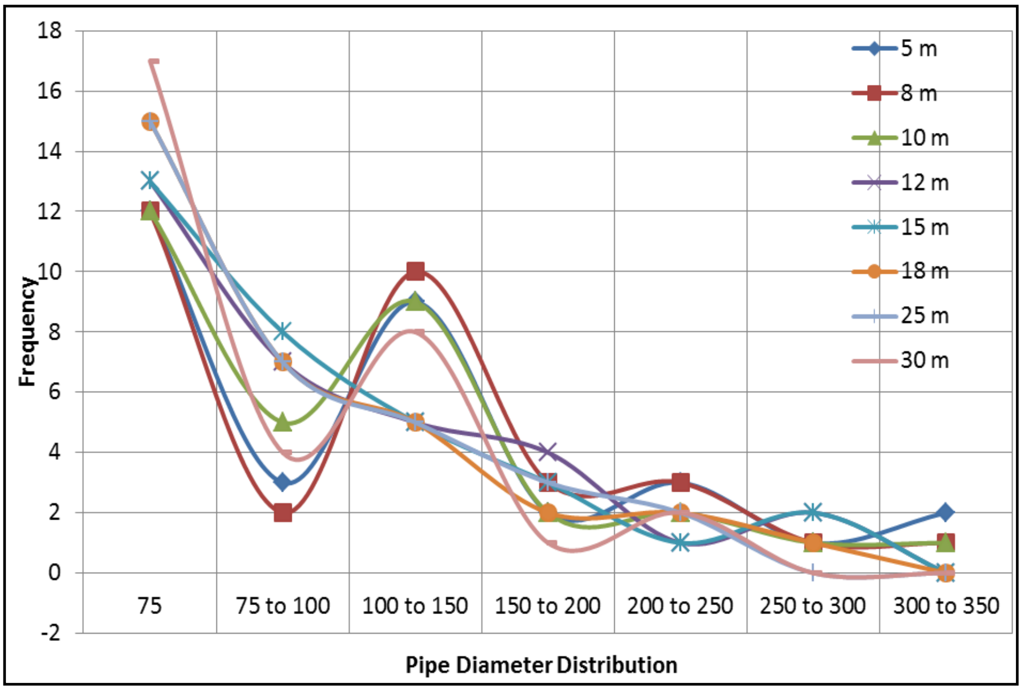

Figure 6: Distribution of Pipe Diameters Based on Minimum Pressure

Table 4: Summary of Optimum Pipe Flows at Various Minimum Pressures

Minimum Pressure (m)		**5**	**8**	**10**	**12**	**15**	**18**	**25**	**30**
Minimum Flows	**Pipe**	RS	KO, LP	DH	RS	RS	HL	PT	PT
	Value (l/s)	0.0074	0.0000	0.0000	0.0257	0.0575	0.0757	0.1014	0.0760
Maximum Flow	**Pipe**	AR							
	Value (l/s)	8.3800 (for all minimum pressure considered)							
Average Flow (l/s)		1.1513 (for all minimum pressure considered)							
Pipes with Negative flow		Nil							

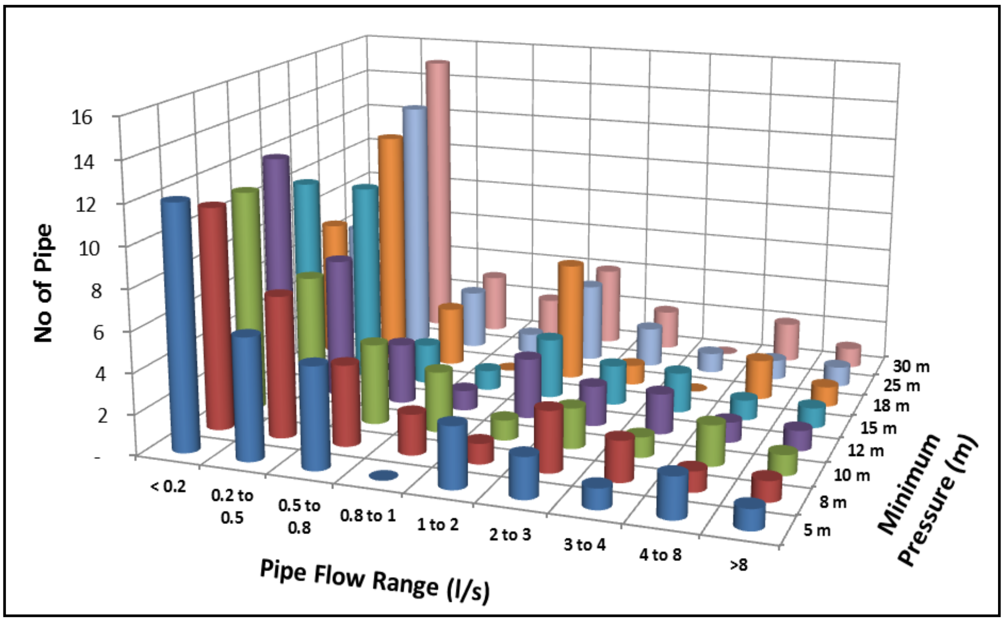

Figure 7: Distribution of Pipe Flows Based on Minimum Pressure

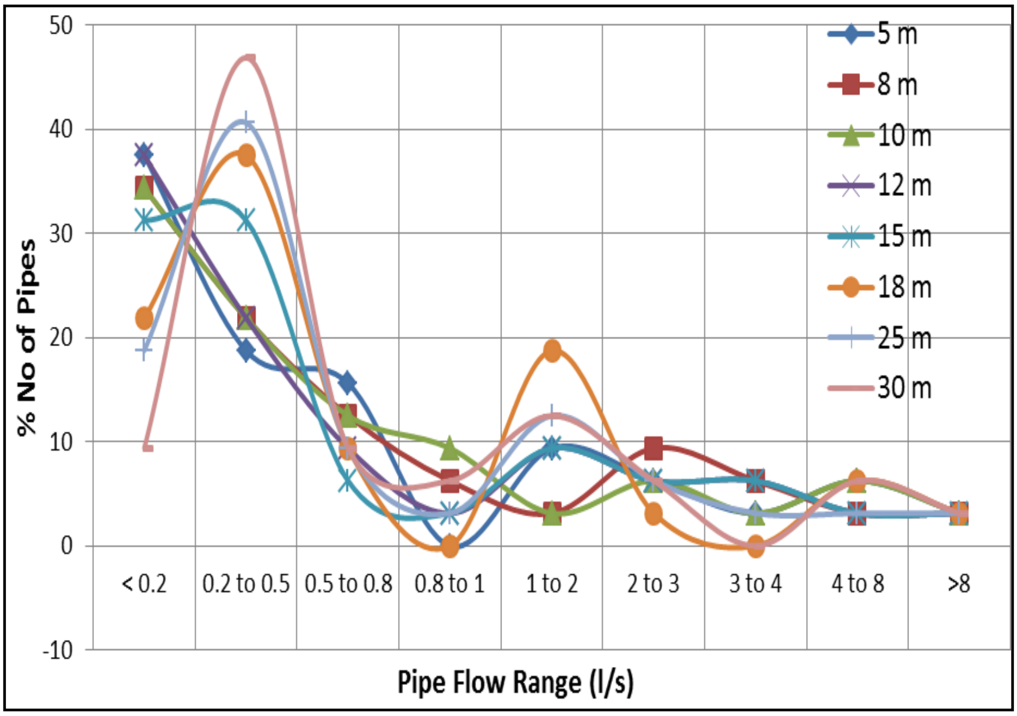

Figure 8: Percentage No of Pipe Flows Based on Minimum Pressure

4.4 Pressure Distribution

The pressure obtained at various nodes depends on the topography of the area under consideration. The pressure increases with increase in depth, hence the lower the elevation of a node, the higher its pressure. The minimum pressure was 5 m at Node A while the maximum pressure was 16 m at Nodes P and T. The average presure in the entire network was 11 m. The values of minimum residual pressure is very adequate for a residential area low rise (bungalow type) buildings. However where higher head is required, the elevation of the reservoir can be increased accordingly. Figure 9 shows the pipe network distribution contour map. The areas served through nodes C, D, H, L, K and G show a pressure between 8 m – 14 m; while nodes F, J, N and M have between 10 m – 12 m. Both ranges are sufficient to serve the households adequately.

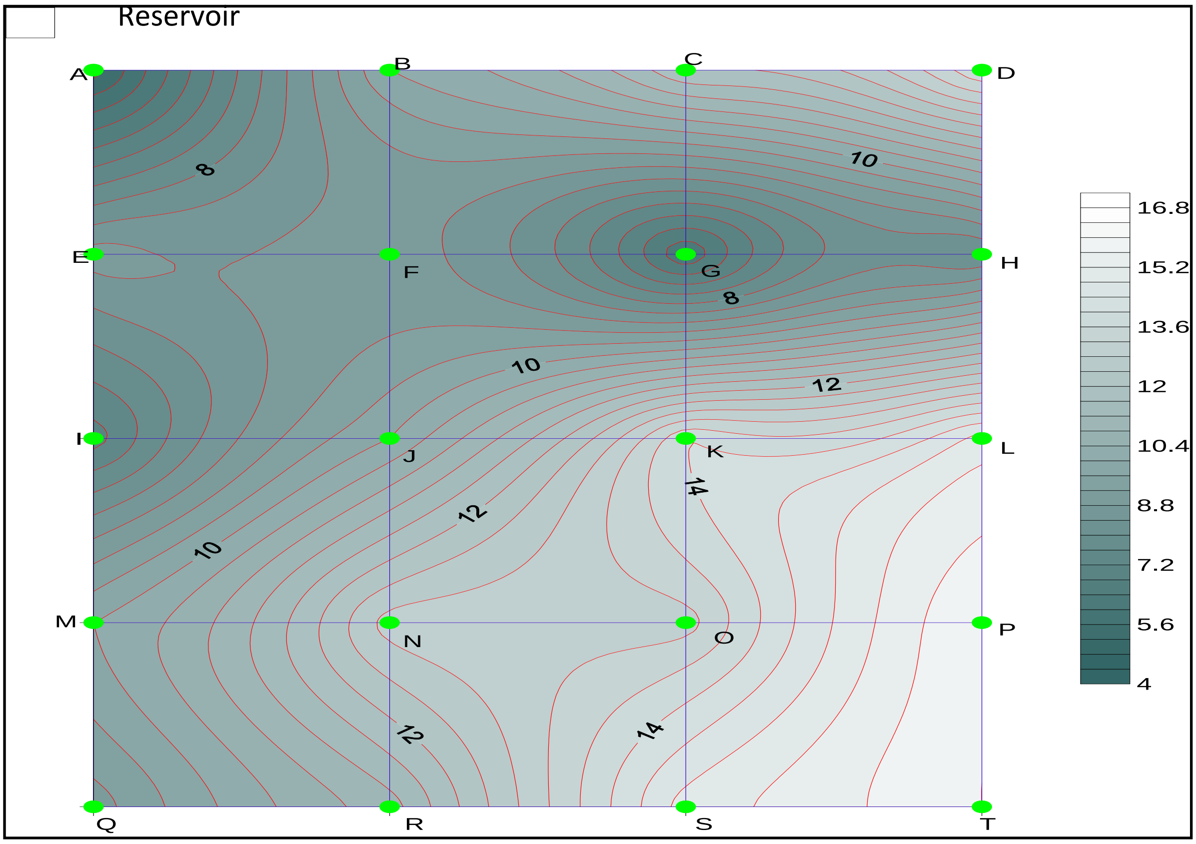

Figure 9: Pipe Network Pressure Distribution

5.0 Conclusion

A Water distribution system connects consumers to sources of water, using hydraulic components, such as pipes, valves, and reservoirs. The engineer faced with the design of such a system, or of additions to an existing system, has to select the sizes of its components. Also he has to consider the way in which the operational components, pumps and valves, will be used to supply the required demands with adequate pressures. The network has to perform adequately under varying demand loads, hydraulic and operational conditions. Operational decisions for these loads are essentially part of the design process, since one cannot separate the so-called design decisions, i.e. the sizing of components, from the operational decisions; they are two inseparable parts of one problem. This work has presented a method for optimizing the design of a water distribution network system using pipe diameter as decision variable under the required demand loading and hydraulic conditions. It has been established that increasing the minimum pressure will lead to the reduction in the required pipe diameter which will in turn reduce the cost of installation.

The optimized system has been examined in such a way that pressure requirement for high rise building (up to 30 m) can still be catered for. The optimum cost of installation as obtained for the study area ranged between 5.7 and 10.5 million naira depending on the minimum pressure required. This modelling approach can be used by engineers and planners to obtain economical pipe sizes for a network designed to serve newly planned layouts.

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