PRAGMATIC APPROACH TO THE COMBINATION AND SELECTION OF TANKS FOR WATER DISTRIBUTION PIPE NETWORK BASED ON PRESSURE SIMULATION

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Abstract
This paper presents a suitable model for appropriate selection and location of tanks based on pressure requirement at demand nodes for municipal water distribution pipe network. The water distribution pipe network data for the supply zone used as case study was obtained for the analysis. Existing tank locations were documented with the help of Geographical Information System (GIS) tools and possible scenarios were determined based on the tank combination using mathematical combination approach. Hydraulic conditions were simulated with EPANET 2.0 software in order to determine output pressure at demand nodes for various scenarios under consideration. The results obtained from the study area showed that the residual pressure is inversely proportion to the reliability index (i.e. as the pressure value increased, the system become less reliable in term of the number of nodes that can enjoy such higher values of the pressure). This will serve as a guide for the proper operation and management of the network distribution system of the city under consideration. The approach is also suggested for evaluating the most efficient strategies for managing nodal pressures in the strengthening of a distribution system or the planning and development of a new one.

Key Words: *Hydraulic, Water demand, Node, EPANET, Supply zone, Scenarios*

Introduction

Water distribution systems, comprising of the water tank, pumps, transmission and distribution supply main, valves and fire hydrants, transport water from a treatment facility to the consumer. Consumers are connected to the sources, through hydraulic components such as pipes, valves, and reservoirs. The objectives of the distribution system is to supply water to every house, industrial plants and public places by means of network of pipes at sufficient quantity and desired pressure, without impairing its quality (Ayanshola *et al.*, 2013). A water distribution system should be able to provide, during its entire life, the required quality of water for the expected loading conditions with the desired residual pressures at all nodes. Despite the installation of distribution reservoirs within different supply zones, many consumers are still left with either no supply or a supply at very low or sometimes negative pressure. Generally, water distribution systems are designed to provide water to consumers at some agreed level of service which is often defined as a minimum level of pressure at the critical point which is the point of lowest pressure in the system (Mckenzie and Wegelin,

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2009). In addition, there may be certain fire-flow requirements which can over-ride the normal consumer requirements (Bhave and Gupta, 2006). According to Jacobs and Strijdom (2009), the stipulation of a minimum pressure requirement during water distribution system (WDS) design is to satisfy the customer. A 'too low' pressure head would not be acceptable and could result in numerous customer complaints. In addition it could lead to operation and maintenance problems, with cost implications if equipment is damaged (e.g. pipe collapse due to negative pressure).

Challenges of providing sufficient water at adequate pressure to support growing demands are becoming more enormous in most cities of developing countries. This results from the failure of the distribution system in many urban cities of developing nations. The strengthening of the existing system or the construction of new ones thus become inevitable. This new developments may sometimes not undergo the necessary analysis and design procedure. One of the most important criteria to be considered when there is a need for a new development is the pressure management. A lot of money has been expended by service providers on this without yielding the desired results. About 80 to 85 percent of the cost of water supply goes into the distribution system (Swamee and Sharma, 2008).

Elevated storage tanks are normally provided within distribution systems to supply peak demand rates and equalize system pressures. In general, elevated storage is more effective and economical because of its ability to reduce pumping requirements. It can also serve as a source of emergency supply since system pressure requirements can still be met temporarily when pumps are out of service. A storage tank can be considered to have failed if it runs dry, or failed to produce desired pressure at demand nodes. It is also possible to describe the reliability or lack thereof of a tank through its failure behaviour as opined by Zyl *et al.* (2008). Increasing the number of tanks and their capacity can reduce pumping costs and invariably improve system reliability if they are effectively positioned and well incorporated into the network.

Water tanks were built at various locations within the metropolis of this study area, Ilorin, central Nigeria, in order to boost the pressure at different locations within the city (Table 1). The idea behind the building of these water tanks was to store the excess water during the low demand periods and release at peak periods. The network will be supplied by water treatment plants together with the water tank. This method aimed at meeting the demand of water network pressure at various nodes. However, most of the tanks were built without thorough evaluation of the number needed for an effective distribution system.

Various techniques exist for the evaluation of a water distribution system. This may include the use of GIS, which is increasingly being co-opted to assign water demand to network nodes based on user classifications such as residential, commercial, industrial, institutional, etc (Filion *et al.*, 2007). Water professionals and researchers had recently found the EPANET software useful in the design, operations and improvement to various water network distribution systems because of its simulation capabilities (Amato, 2014; Adeniran and Oyelowo, 2013). These tools and approach was employed in this work in order to effectively carry out this assignment.

This study therefore seeks to evaluate the existing system based on pressure simulation in order to establish the best

combination of tanks that will be most effective in the network distribution system. This will serve as a guide for a proper operation and management of the network distribution system of the city under consideration. The approach will be useful for evaluating the most efficient strategies for managing the network in order to reduce high inequality nodal pressure in a supply zone before embarking on strengthening of network distribution system by introducing tanks.

Method and Materials
Study Area

A zone of supply was selected within the Ilorin township water distribution network. Ilorin, the capital city of Kwara State, Nigeria is located between latitudes 8° 25′N and 8° 32′N longitudes 4°30′E and 4°41′E (Figure 1). It presently occupies an area of about 89 Km^2 (Adeleke, 2010). According to Ayanshola *et al.* (2013), the population of the city was estimated to be 777,667 in 2007. The zone selected for this study is within the metropolis and is shown in a modified downloaded google earth map described by Figure 2.

Data Collection

A digital map of the pipe layout for the town was obtained from the Kwara State Water Board (KSWB), who is the service provider. Information regarding features such as link diameter, length, nodal elevation, tank location and height and all other fittings and appurtenances that constitutes the distribution system was collected. Population data was obtained from the National Population Commission.
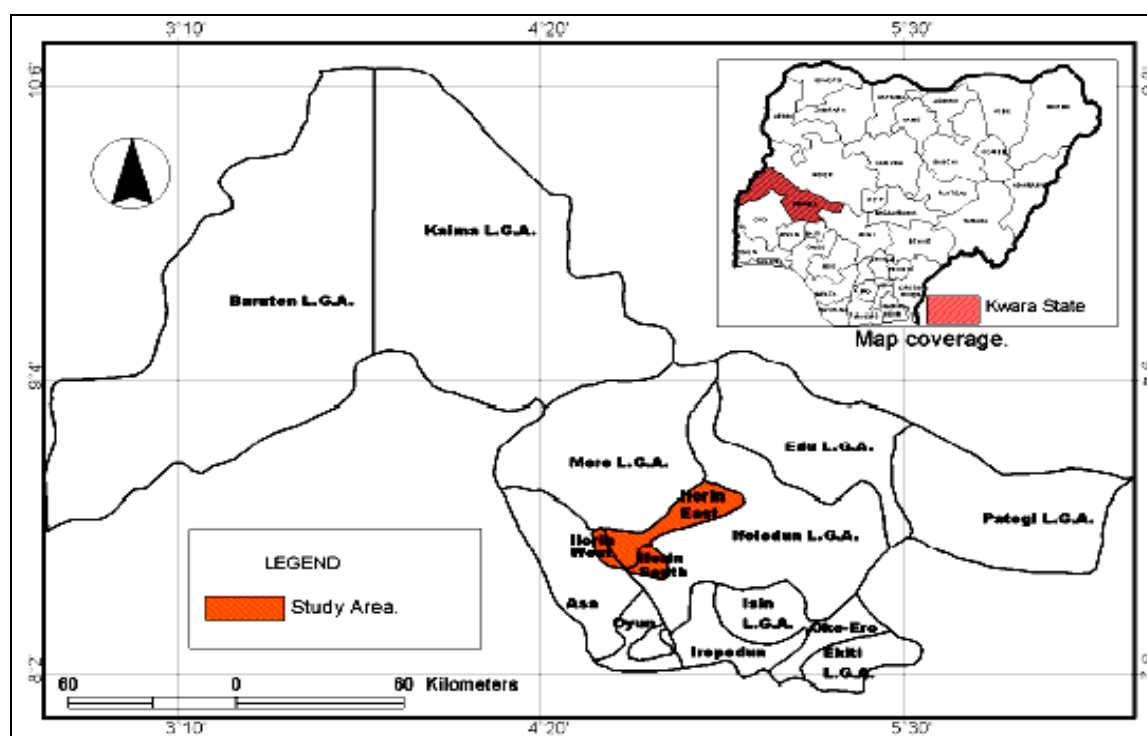

Figure 1: Map of Kwara State Showing the study Area [Source: Adebimpe, 2011]

The supply system in Ilorin is sourced from the Agba, Asa and Moro Rivers. Reservoirs built on these rivers have a total reservoir storage capacity of about 49,100,000 m^3 with total treatment plant full capacity operation of about 82,276

m^3/day. Water pumped from Asa, Agba and Sobi clear water tanks are normally stored in distribution/service reservoirs from where they are delivered by gravity to the various demand areas. Before 1999, seven service reservoirs were connected to the network. Another six overhead steel tanks were introduced in 2001 to enhance the distribution system out of which two are replacements. The location, capacities and the status of the 13 service reservoirs in Ilorin Township are as shown in Table 1 (Ayanshola and Sule, 2006).
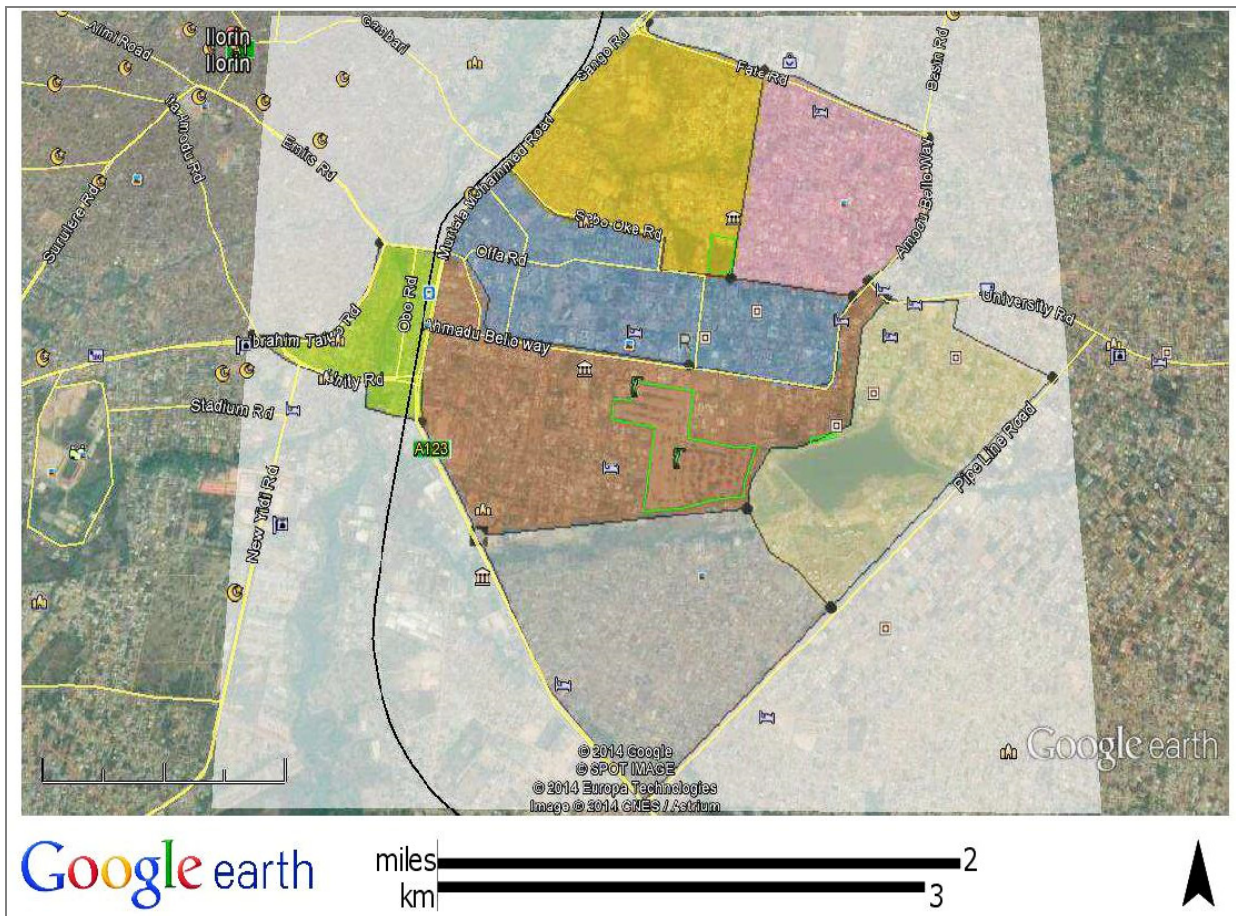

Figure 2: Modified Google Earth Map Showing the study Area

The distribution system in the metropolis is primarily dictated by street patterns, topography, water demand and location of treatment and service reservoirs. For Ilorin Metropolis, the pipe network is made up of Asbestos Cement (AC), Cast Iron (CI), Galvanized Iron (GI) and Unplasticized Polyvinyl Chloride (UPVC) pipes ranging from 50 mm to 600 mm diameter and cover a total distance of about 337.542 km. In recent time, there were additional pipelines ranging from 100 mm to 800 mm diameter made of D.I. and UPVC which cover a total distance of about 107.869 km to upgrading the system. Out of the total length of the pipelines, 79.017 km are new pipelines either for rising main or distribution network while the remaining length of about 28.852 km were used to replace some of the existing AC pipelines within the township.

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Table 1: Location, Capacities and Status of Distribution Tanks in Ilorin Township

S/no	Tank type	Location	Source	Capacity (m³)	Status as at 2006	Present status
1.	Underground Concrete	GRA/Fate	Agba	4,600	In use	In use
2.	Overhead Steel	GRA/Fate	Agba	454.45	In use	In use
3.	Eastern Ground Level Concrete	Tanke	Asa	10,000	In use	In use
4.	Western Ground Level Concrete	Adewole Estate	Asa	10,000	In use	In use
5.	Overhead Concrete	Former UITH	Asa	4,600	In use	In use
6.	Overhead Steel	Olanrewaju Estate, Offa Garage	Agba	227.27	Replaced	Not in use
7.	Overhead Steel	Olanrewaju Estate, Offa Garage	Agba	454.45	In use	In use
8.	Overhead Steel	Oke Sobi, Sobi Road	Sobi	227.27	Replaced	Not in use
9.	Overhead Steel	Oke Sobi, Sobi Road	Sobi	454.45	In use	In use
10.	Overhead Steel	Central Mosque	Asa	227.27	Not in use	removed
11.	Overhead Steel	Central Mosque	Asa	454.45	In use	Removed
12.	Overhead Steel	Unity-Taiwo Junction	Agba	454.45	In use	In use
13.	Overhead Steel	Kwara Hotel	Agba	454.45	In use	In use

Data Analysis
Population and Water Demand Calculation

The projected population of Ilorin for the year 2020 is 1,185,033 from equation 1. This is based on the 1996 figures provided by the National Population Commission and a geometric growth rate of 2.83:

$$Pn = Po(1+r)^n \qquad 1$$

Where Pn = Present population, Po = Initial population, r = growth rate and n = number of year.

The NPC did not provide a breakdown of the population of each area within the metropolis and thus the population covered by each loop was based on the assumption that the town has an equal population density for all areas. This means that the population of an area is dependent on the area enclosed by the loop. For an estimated area of 89 km², the population density is 13 people per square metre. Table 2 shows the areas and corresponding population of the loops under consideration while Table 3 shows the corresponding pipe lengths. All the pipes are 300 mm in diameter since they are distribution lines. A roughness coefficient of 140 was used for the analysis since the pipes are made of Asbestos Cement (Steel and McGhee, 1984). Total pipe length for each loop was also determined in order to estimate demand in each of the loop shown in Figure 3. All the parameters were determined using the available data from KSWB with the aid of GIS and google earth map.

Table 2: Water demand along the pipes

Loop	Pipe composition	Population	Area (m²)	Total pipe length (m)	Average demand (l/s)	Peak demand (l/s)	Consumption per length (x 10⁻³ l/s/m)
1	JK, KP, PQ, JQ	18060	1389.22	6101.26	25.08	50.17	8.22
2	KP, KL, LM, MO, OP	13860	1066.14	3487.25	19.25	38.50	11.04
3	IJ, IR, LR, KL, JK	17561	1350.84	7095.48	24.39	48.78	6.87
4	FI, DF, DE, EN, MN, LM, LR, IR	25385	1952.68	9462.00	35.26	70.51	7.45
5	BE, EN, AN, AB	16191	1245.50	6038.85	22.49	44.98	7.45
6	BE, BC, CD, DE	19834	1525.69	5950.78	27.55	55.09	9.26
7	FG, GH, HI, FI	6030	463.82	4244.86	8.37	16.75	3.95

Table 3: Pipe Lengths

Pipe ID	Length (m)	Pipe ID	Length (m)
AB	2027.93	GH	1231.66
BC	1602.54	HI	448.92
CD	1808.23	IJ	716.69
DE	1757.20	JK	1945.61
BE	782.82	KP	1379.97
EN	1816.15	OP	1204.68
AN	1411.96	MO	1127.35
MN	134.52	KL	849.22
LM	131.716	JQ	1397.96
FI	1099.11	PQ	1378.73
DF	939.34	LR	1880.52
FG	1465.16	IR	1703.45

Tank Combinations and Scenarios

The general methodology of this hydraulic simulation and analysis was to examine the pressure variations of different service reservoirs combinations within the zone under consideration. The possible number of combination for the service reservoirs in a service zone is given by:

$$nCr = \frac{n!}{(n-r)!r!} \qquad 2$$

Where: C is the possible number of combination without repetition, n is the total numbers reservoirs in the supply zone and r is the number of reservoirs to be combined at a time. Based on equation 2, the total number of possible scenarios (combinations) is 31 and the breakdown is in Table 4.

Table 4: Possible tank combinations

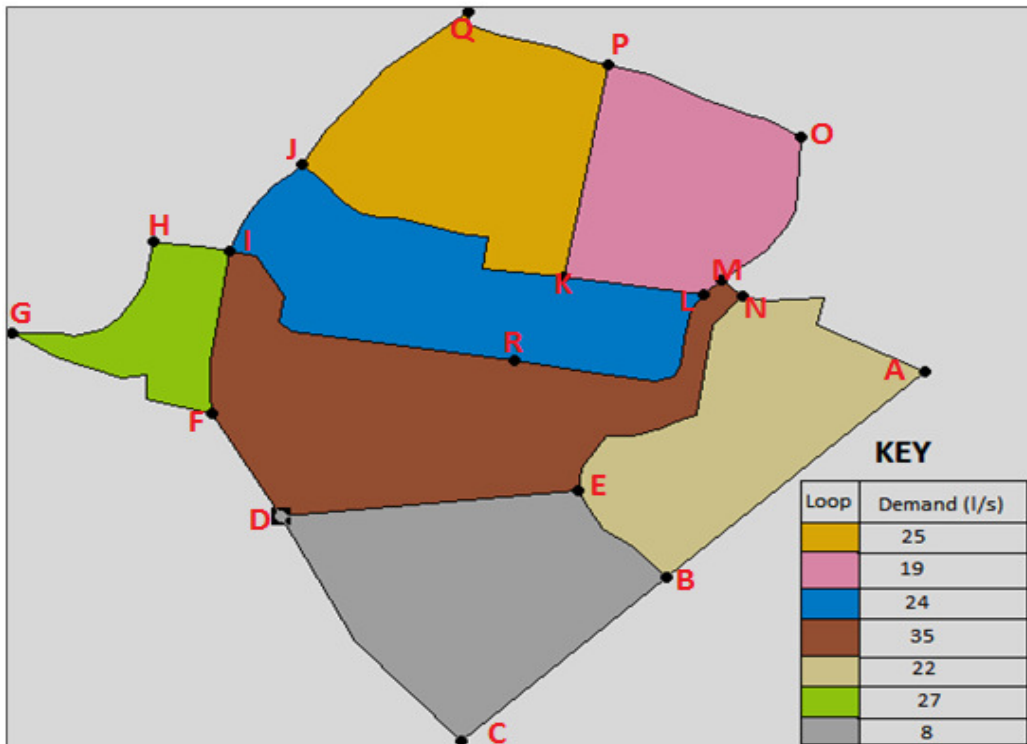| No of tanks (n)	No Combined (R)	Possible No of Combinations (C)
5	1	5
5	2	10
5	3	10
5	4	5
5	5	1
TOTAL		31

Figure 3: Pipe Network Distribution Zone Showing Demand Variation

Pressure Simulation and Analysis

Pressure was the primary hydraulic parameter analysed using EPANET 2.0 software to identify the best combination of the service reservoirs to serve a distribution zone effectively. The target minimum and maximum residual pressure are 5 m (7 psi) and 70 m (99 psi) respectively during peak hour demand conditions. This range of residual pressure has been found to be adequate for municipal water supply system (Ayanshola and Sule, 2006). EPANET uses the same numerical engines as WaterCAD and other commercially available software. Several commercial simulation models are available, but in this study EPANET 2.0 was used to perform hydraulic simulation. EPANET 2.0 fulfils the requirement of calculating nodal pressures and its source code is available free of cost in the public domain. It can also be applied to large water distribution networks with unlimited pipe numbers (Adeniran and Oyelowo, 2013; Avesani *et al*., 2014).

EPANET's hydraulic simulation model computes hydraulic heads at junctions and flow rates through links for a fixed set of reservoir levels, tank levels, and water demands over a succession of points in time (Ayanshola, 2013; Ayanshola *et al.*, 2013). The principle of EPANET network analysis is based on the following two conditions (Adeniran and Oyelowo, 2013; Avesani, *et al.*, 2014): the algebraic sum of the pressure drops around a closed loop must be zero, i.e. there can be no discontinuity in pressure and; the flow entering a junction must be equal to the flow leaving the same junction; i.e. the law of continuity must be satisfied. Based upon these two basic principles, the pipe networks are generally solved by the method of successive approximation (Sil *et al*., 2012).

Results and Discussion

The analysis was performed for 31 scenarios earlier mentioned. The results showed that few scenarios generated the desired level of service in the supply zone. Figure 4 shows the percentage numbers of nodes with positive residual pressure for various scenarios under consideration. The study showed that about 38 % of the total number of scenarios has up to 50 % nodes with positive residual pressure value. Only three cases were found to have 100 % positive pressure while the remaining 28 cases generated negative pressures in more than one node. Such cases with negative pressures at node of demands should be avoided because faecal organisms and culturable human viruses may be present in groundwater adjacent to a pipeline and drawn into network (Chambers *et al.*, 2004).

Further analysis was carried by selecting the best 5 combinations ('1 and 4', '1 and 5', '1, 2 and 4', '1, 4 and 5' and '1, 2, 4 and 5') in order to establish the index of their reliability as shown in Figure 5. These are the cases that had up to 80 % positive pressure at their nodes. The relationship that exists shows that the residual pressure is inversely proportion to

the reliability index as shown in Figure 5. As the pressure value increased, the number of nodes that can enjoy high values of pressure reduces. However, the combination of Reservoirs '1, 4 and 5' yielded the best result as it produces the highest reliability index up to minimum of 5 m residual pressure. This was followed by the combination of '1, 2, 4 and 5' while the combination of '1 and 5' is the least desirable of the possible combinations.

The best three scenarios were furthered evaluated as shown in Figures 6. The scenario that made use of the combination of Tanks 1, 4 and 5 yielded best result because it spread between the pressure of 5 m and 30 m. This is a good pressure distribution within the zone under consideration. In this scenario, few nodes will still enjoy as much as 50 m residual pressure. A scenario with Tanks 1 and 4 shows that no nodes have a pressure range between 15 m and 30 m even though few nodes enjoy higher residual pressure that tops 49 m. For the '1, 2, 4 and 5' combination, there is no even distribution of pressure among the nodes. Most nodes experience higher and lower residual pressure.
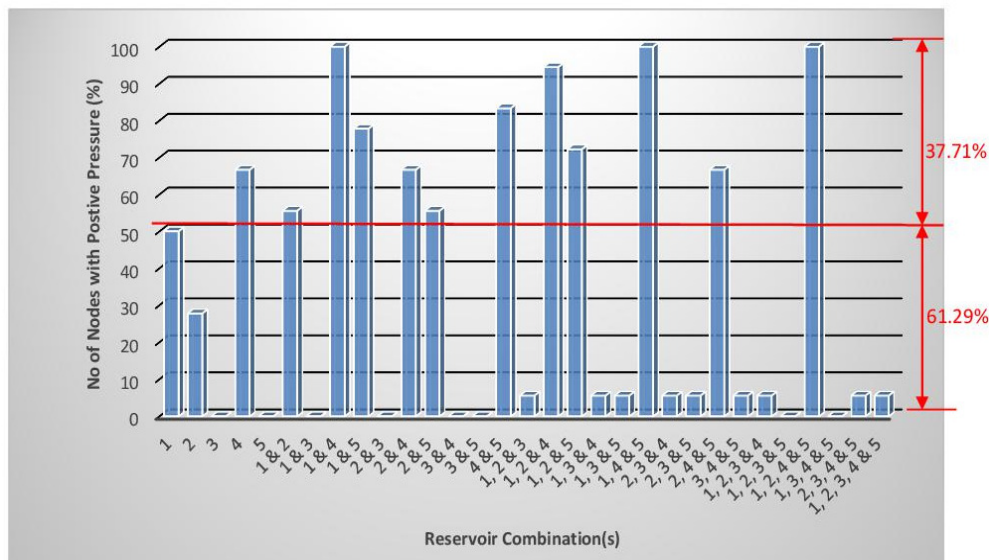

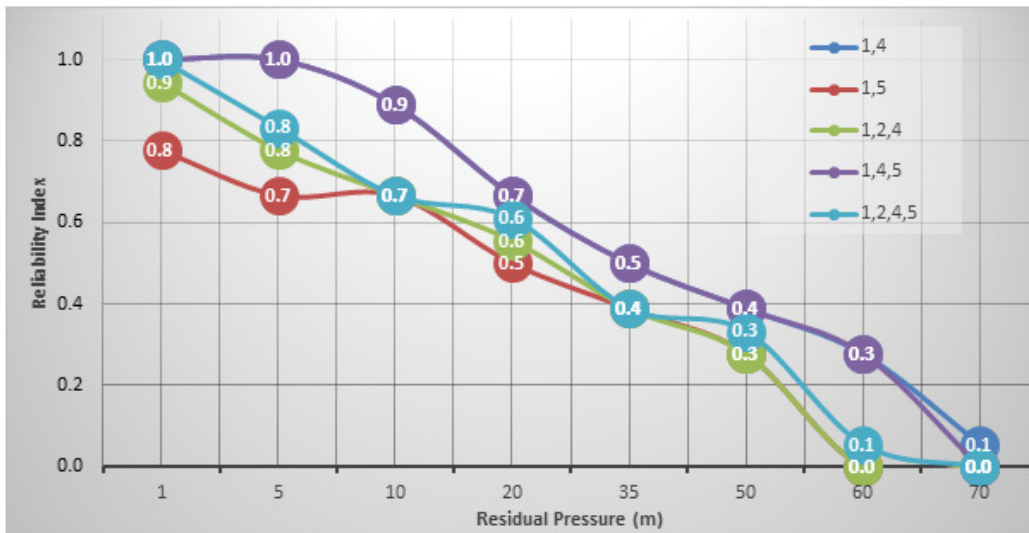Figure 4: Variation of Percentage Number of Nodes for Various Tank Combinations

Figure 5: Reliability Index and Pressure Variation

Figure 7 shows the spatial variation of pressure simulated for the scenario with Tanks 1, 4 and 5 combined. The loops around nodes A, B, C and D has the highest pressure and it kept on reducing towards nodes O, P and Q. However, they all fall within the minimum desirable pressure for a municipal water supply system.
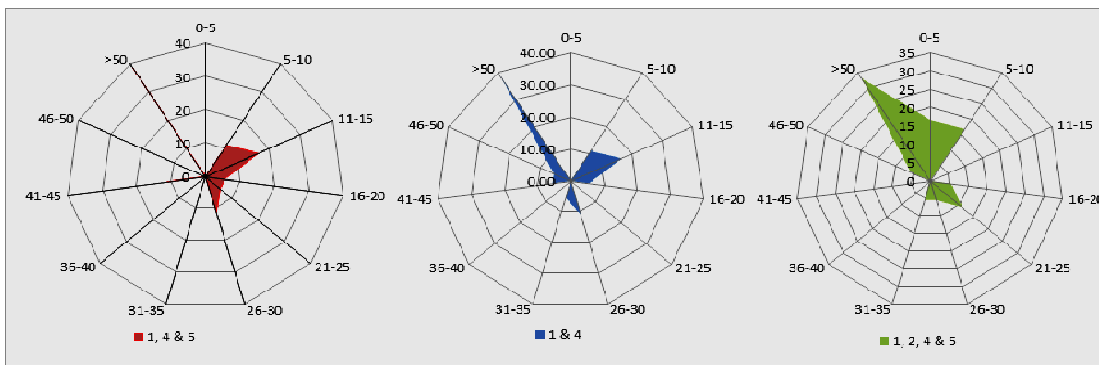

Figure 6: Pressure Variation with Percentage Number of Nodes for the Best Three Identified Combinations of Tanks
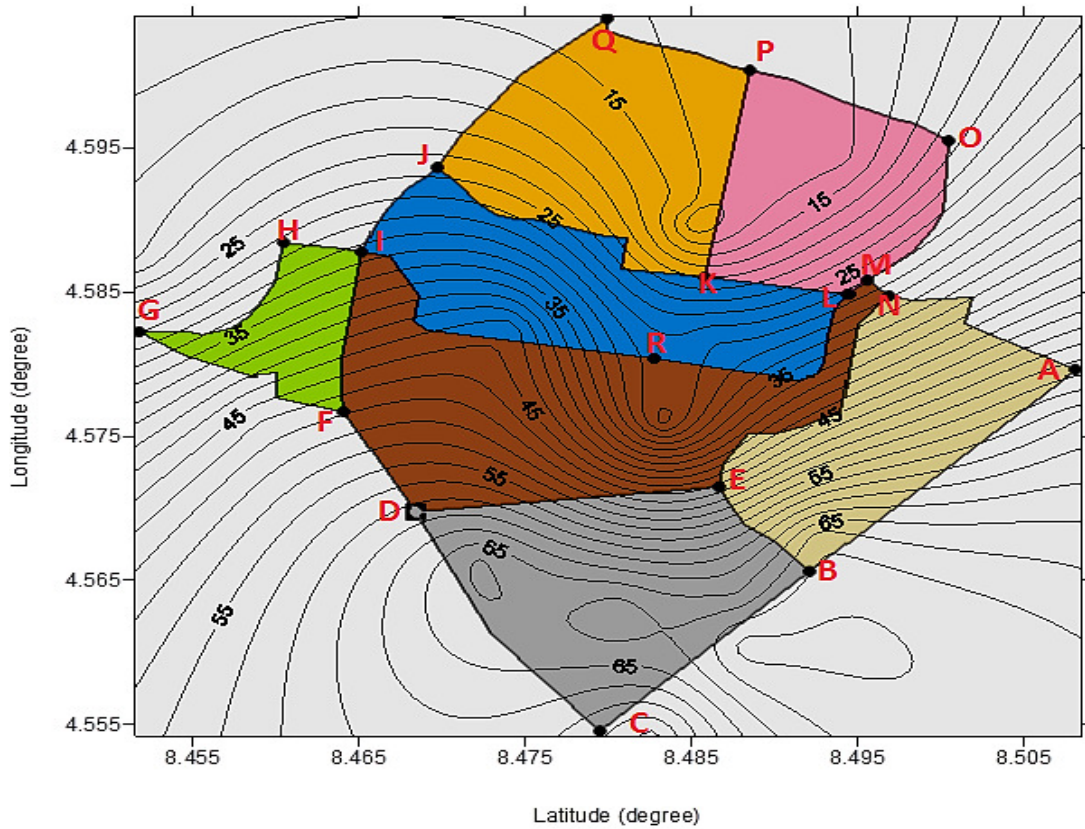
Figure 7: Spatial Variation of Simulated Pressure for Various Demand Zone

Conclusion

This study evaluated an existing system of location of storage reservoirs based on pressure simulation in order to establish the best combination of reservoirs that will be most effective in the network distribution system. The results obtained from the study area showed that the residual pressure is inversely proportion to the reliability index. This finding will serve as a guide to the proper operation and management of the network distribution system of the city under consideration. This approach will also find relevance when embarking on the strengthening of a network distribution system by the introduction of overhead reservoirs or in new developments.

Acknowledgement

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