Optimization of Water Distribution System Using WaterGEMS: The Case of Wukro Town, Ethiopia

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Abstract

This study was aimed to optimize the designed water distribution system in the Wukro town using WaterGEMS model. The Darwin Designer in WaterGEMS was applied for finding optimal pipe diameter to supply adequate quantity of water at satisfactory pressures to the end users. In the WaterGEMS model, the Darwin Scheduler of daily pumping operations tools also used for optimal control and operation of pump systems. The WaterGEMS model was implemented in water distribution networks which include 117 pipes (40.67km), 99 demand nodes (equivalent to 50480 end users) that are spread across a hilly area over a 1989m to 2046m elevation gradient. The model was calibrated at the selected nodes within very good performance. The results have shown that the maximum pressure before optimization is 31.1m and after optimization increased to 38.1m, the minimum pressure on the former is 7.9m and 16m later during peak hour demand. Comparison of results showed that the optimized networks reduce the cost by 9.6% than those of before optimization networks by traditional hydraulic. In addition to this, the optimal tanks filling/emptying arrangement decreased the daily cost of energy consumptions by 12.5% compare as a currently scheduled pump. The finding of this study indicated that the WaterGEMS model is a promising approach for optimal sizing of pipes in design water distribution networks and pumping operational schedules.

Keywords: Water Distribution Network, Genetic Algorithm, Pipe Diameter, Energy consumption, Darwin Designer, Darwin scheduler

1. Introduction

Water distribution systems (WDSs) are vital infrastructure for supplying water from the source to the end users. It has been designed commonly to provide an adequate amount of potable water with sufficient pressure at the consumer, where the water demand is required namely the residential, industrial, institutional, and commercial (Elsheikh *et al*., 2013). The WDS infrastructure has been constructed and developed for hundreds of years across in the world on the population growth and urbanization (Ramos *et al.*, 2018).

Several scholars have been optimized the WDS through modified the pipe diameter, energy consumptions of the pump, and location of the reservoir (Bello *et al.*,2015; Ramos *et al.*, 2018; Boano *et al.*, 2015). As a result of this, an optimized WDS should be implemented to minimize the total cost of the project and also satisfies required water flow velocities and pressure on the nodal. The numerous optimization methods are studies in the worldwide such as GA (Djebedjian *et al.* 2006; Guc, 2006), linear programming (Samani and Zanganeh, 2010; Bello *et al.*, 2015) nonlinear programming and dynamic programming (Keles, 2005; Akdogan, 2005) talk too much time for computations and need more powerful computer (Guc, 2006).

However, many of these software's are complex and difficult for technical application in the real networks of developing countries, where few number of skill manpower's. Recently, it is easier to model and optimization theories with the help of WaterGEMS (Water Geospatial Engineering Modeling System) software. This software is suitably used in hydraulic model to optimize the WDS in worldwide using the Darwin Designer and Darwin Scheduler tools based on the Genetic Algorithm (GA) (Switnicka *et al.*, 2017). It deals with the extended period simulation and linked with optimization software.

Dhumal *et al.* (2018) investigated continuous water supply systems designed and developed by using WaterGEMS software effectively. Model calibration was performed to minimize the difference between the observed and simulated pressures. Besides, it optimizes the distribution network on the source of performance and cost. Finally, the authors concluded that WaterGEMS is more advanced computing systems to hydraulic modeling and designing software with the help of different applications on the software.

Sumithra *et al.* (2013) compared the skill of the WaterGEMS and LOOP software. The authors found that WaterGEMS is more reliable, efficient and change can be done very easily. Besides, WaterGEMS found extremely user-friendly with a variety of hydraulic and graphical analysis options. It is also less time consuming for the renovating and reanalyzing the network. Finally, the authors concluded that the WaterGEMS is more advantage for modeling and optimization will improve the WDS

Darwin Designer is an advanced tool in WaterGEMS to finds minimum cost or maximum benefit designs and

rehabilitation strategies, form on capital investment, reposition cost, and pressure and velocity constraints. Since the tool is available in WaterGEMS includes GA optimization engines for automated calibration, design, and rehabilitation. Most of the experienced researchers are concluded to manage infrastructure capital cost (Bentley, 2014).

Bello *et al.* (2015) Optimized of the designed water distribution system in case of Dukku town, Nigeria. The author collected the water distribution network data. The WDS optimized the pipe diameters and a decrease in total cost of pipes. The initial cost was reduced to 7.15% and hydraulic properties of the entire distribution network were improved. The maximum pressure pre-optimization is 14.79m and post-optimization increased to 15.71m. The final optimized WDS were improved the flow rate, velocities, and headloss to addresses the problem of the water shortage of the studied area.

Darwin Scheduler is also state of the advanced tool for optimizing pump operation that works by using GA optimization to control nominated pumps during an EPS (Ali, Abozeid, Darweesh, & Mamdouh, 2015). The GA optimization technique works by developing near-optimal solutions over generations of trial solutions. One problem with this fact is that EPS simulations can be time-consuming, especially for larger or more complicated models, and therefore run times for Darwin Scheduler can be correspondingly long. These best practices and tips offer suggestions and recommendations for using Darwin Scheduler in order to get the best performance and results out of the tool (Bentley, 2014).

Ali *et al.* (2015) optimized pump schedule to save energy using WaterGEMS in case study El-Mina town in Ghana. The authors compare the optimized and existing pumping schedule using the Darwin scheduler for 24 hours difference with the number of opening and closing the pump in one hour. Pump schedule was achieved three common constraints which are pressure, velocity and tank constraints. Finally reduced the daily pump energy consumption by 7.5% (676.6 KWh). The objective of the study is to optimize the WDS of Wukro town by modifying the pipe diameters and energy consumption for the least cost design using WaterGEMS software.

2. Material and methods

2.1. Description of the study area

Wukro town lies in the Tigray regional state of Ethiopia. It is far from the capital city about 830 kilometers northern of Addis Ababa and about 47 km north of Mekelle capital city of the regional state of Tigray. The geographic position of Wukro is found within 13°46'N to 13°48'N and 39° 35'E to 39°37'E as shown in Figure 2.1. The altitude elevation of the study area is also found between 1989m to 2046m amsl and mean monthly temperature often observed between 8.92^0C to 25.9^oc throughout the year in the last ten years. The total population of the study area is a population of 30210 in the year of 2007 as per the Central Statistical Agency (CSA, 2007) report.
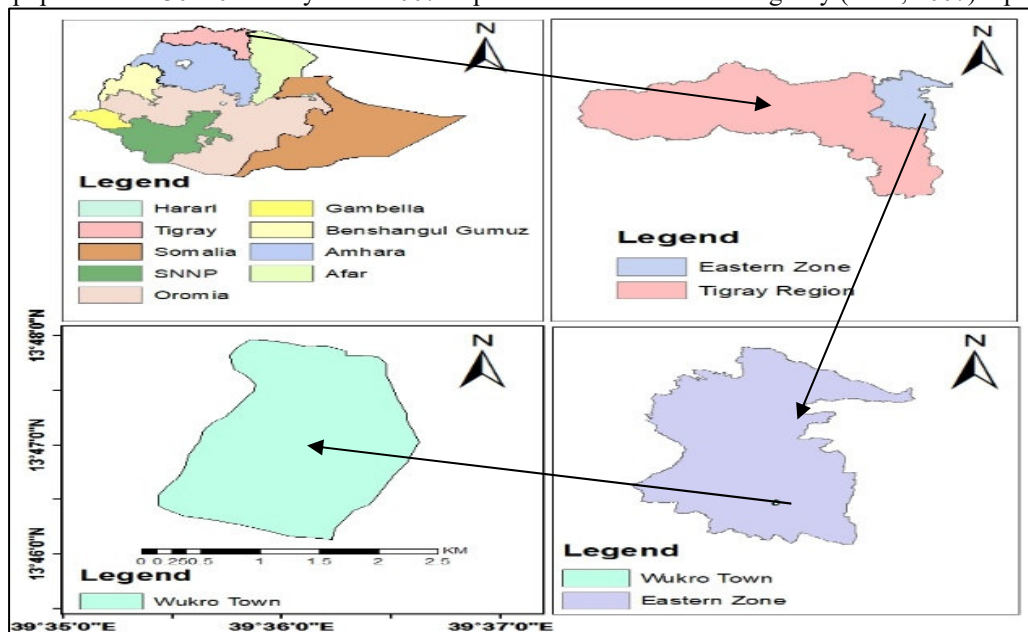

Figure 2.1 Location of case study area

2.2. Wukro water supply system

The existing water source of the Wukro town was supplied water from seven boreholes which have a combined average water production capacity about 43.5l/s, but the delivered water to the consumer is 34.8l/s as per collected from Wukro Water Supply and Sanitation Service (WWSSS). It was yielded water from boreholes which are

located around the town border and Abraha Weatsbeha (8km) from the center of the town.

WDS of the town was examined through a typically branched and looped network made of a mixture of DCI, HDPE, and GI pipe materials with sizes range from DN 40mm to DN 200mm starting from borehole to every consumer with a total estimated length of distribution lines is covered about 57.2km.
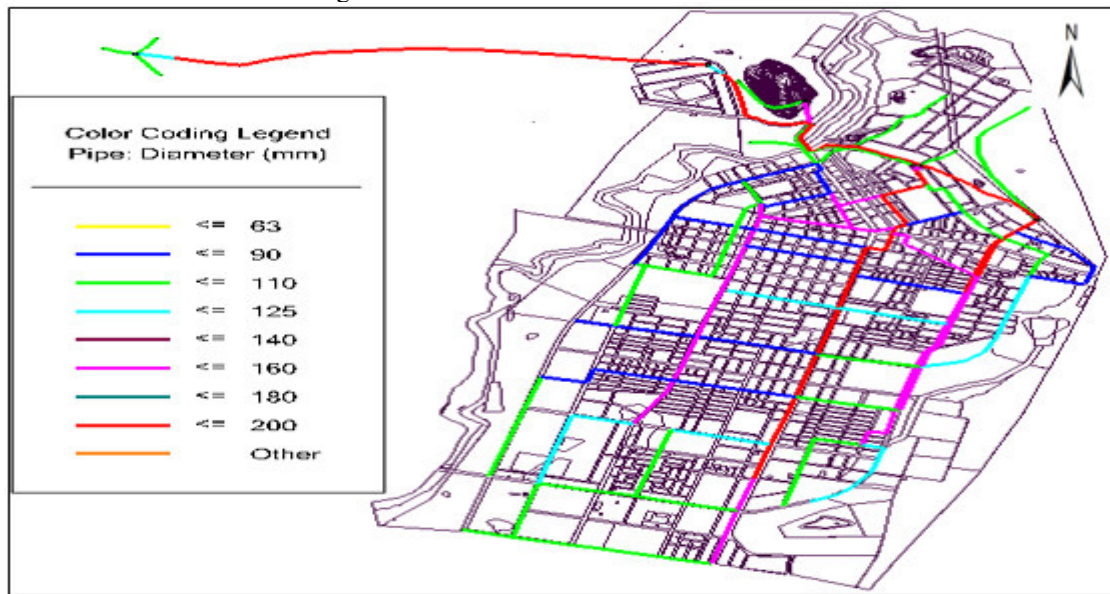

Figure 2.2 Before optimization of pipe diameter

2.3. Non-revenue water

The water balance method was used to estimate the water loss in the distribution system determined by Eq. 2.1. This method calculates the total water loss in the distribution system from total flows of water (Al-Bulushi *et al.*, 2018; Gisha *et al.*, 2016). This describes the difference between inflows to the distribution system and all types of water consumption.

$$NRW = \frac{W_{tot.\ prod} - W_{tot.\ cons}}{W_{tot.\ prod}} \qquad (2.1)$$

Where NRW is Non-revenue water (%); $W_{tot.prod}$ is total water produced(m^3/yr); and $W_{tot.cons}$ is total water consumed (m^3/yr).

2.4. Water demand variations

EPS analysis was used to provide a specific pattern for giving demands with respect to hourly, and daily variation in water demands.

$$PF = \frac{Q_{max}}{Q_{av}} \qquad (2.2)$$

Where PF is peak factor in a daily hour demand; Q_{max} is maximum hourly demand (m^3/hr); and Q_{av} is average daily demand (m^3/hr).

2.5. Model skeletonization

The model skeletonization process was done in WaterGEMS using the skelebrator skeletonizer tool. According to USEPA (2005) the skeletonization of pipes are study adopted by conditions as follows: at least 50% of total pipe length of the distribution, at least 75% of the pipe volume in the distribution system and all DN 200mm, DN 300mm and larger pipes that connect pressure zones, influence zones from different sources, storage facilities, major demand areas, pumps, and control valves to be significant transport of water. Thus, as shown below Table 2.1 the above requirement was performed to skeleton the pipes based on length and volume of pipes in the WaterGEMS model before and after skeletonization skeletonized from the DN 90mm to the larger pipes DN 200mm diameter.

Table 2.1 Length and volume of pipes in the model before and after skeletonization

	Before model skeletonization	After model skeletonization	Percentage of skeletonization (%)
Total length of pipes (m)	57222	40670	71.1
Total volume of pipes(m³)	687.2	638.6	92.9

2.6. Model calibration

Model calibration was determined based on the results of model pressure and measured pressure in the selected nodes have been used for calibration. According to AWWA (2012) the percentage of measurement nodes satisfied the essential minimal amount of measurement points (2% of all nodes) for the designing and operation purposes. Pressure gauge was measured the pressure of water at five nodes (J-5, J-23, J-60, J-86, and J-132) from 135 nodes in particular hours (6:00 AM, 9:00 AM, 6:00 PM and 9:00 PM) for five days to check the simulation results. The model performance was evaluated using objective functions such as Coefficient of determination (R^2), Mean Error (ME) and Root Mean Square Error (RMSE).

2.7. Optimization of WDS

2.7.1.   Optimization of pipeline system

Darwin designer was used to optimize makes to solve optimization problems with very large solution spaces which cannot be solved using more conventional optimization methods within the GA parameters (Ali *et al.*, 2015).

The optimization of pipe diameter was done to reduce the cost pipe based on nodal demand 82.1l/s as analyzed in the system. The existing before optimization of the Wukro town distribution system have presented in Figure 3.4 along a different type of pipe diameter

2.7.2.   Pump optimization

Pumping schedule time was optimized based on the current water production to become an advantage for future pumping system. A number of other studies have also demonstrated the effectiveness of Darwin Scheduler operational optimization of WDSs (Ali *et al.*, 2015; Switnicka *et al.*, 2017; Ramos *et al.*, 2018).

The optimization pumps were defined in terms of tank level controls (Zyl *et al.*, 2004). Tank level controls activate the control actions in the distribution system when tank water levels reach certain predetermined values and widely used in practice based on switch on and off the pumps to schedule time period for typically 24 hours. In such a way the distribution system requires 8 pumps in the 8 pump stations for each pump which connect the pipe from source reservoir and collection chamber.

2.7.3.   Total cost objective function

The objective function of the study was to the optimal design problem is usually the minimization of total costs of pipe diameter and energy consumed by the pumps, in every operational time using the Darwin Designer and Darwin Scheduler optimizer in WaterGEMS used as below:

$$\text{Minimize} \qquad C_T = C_{pp} + C_{pu} \qquad\qquad (2.3)$$

Where C_T is the total cost of the solution (ETB); C_{pp} is the cost of pipe diameter (ETB); and C_{pu} cost of pump energy consumption (ETB).

$$C_T = \sum_{i=1}^{N_{pp}} C_i L_i + C_p \sum_{t=1}^{N_{pu}} E_t \qquad\qquad (2.4)$$

Where C_T is the total cost of the solution (ETB); C_i is the cost per unit length of the ith link (ETB/m) with diameter D_i (mm); L_i is the length of pipe i^{th} (m); C_p is the cost of energy consumption (ETB/KWh); E_t is the electric energy consumed at time interval t (KWh); N_{pp} is the number of pipes; and N_{pu} is the number of pumps.

For this study, according to Ethiopian standard MoWR (2006) pressure and velocity are constrained on the 15m to 70m and 0.3m/s to 3m/s respectively, and the tanks constrained on the water level based on switching the pumps were used to optimize the cost formulated as follows:

A.   Pressure constraints

Pressure constraints were the design and hydraulic constraints of the pressure head bounds at each nodes determinedvas:

$$H_{j\ min} \leq H_j \leq H_{j\ max} \qquad\qquad (2.5)$$

Where H_j is the pressure head at node j with t time; and $H_{j,\ min}$ and $H_{j,\ max}$ are the minimum and maximum allowable pressure heads at node j.

B.   Velocity constraints

The operational constraints contained velocity limitations at each pipe was calculated by:

$$V_{i\ min} \leq V_i \leq V_{i\ max} \qquad\qquad (2.6)$$

Where V_i is water velocity of pipe i at time t; $V_{i\ min}$ and $V_{i\ max}$ are minimum and maximum water velocities in pipe

i at time t respectively.

C.   Tank constraints

A storage tank constraint in a WDS was operated within a minimum and a maximum allowable water level. Storage tank used to control pump system the flows of water to on/off the pump system determined as:

$$Y_{i\,min} \leq Y_{i\,t} \leq Y_{i\,max} \tag{2.7}$$

Where $Y_{i\,min}$ and $Y_{i\,max}$ is minimum and maximum water levels for tank i; $Y_{i,t}$ is balanced water levels of tank i at t time.

D.   Pump switching constraints

Energy cost is reduced by turning a pump on and off many times during a control period (Ramos *et al.*, 2018). However, the more frequent a pump switches on and off the greater resulting pump maintenance cost due to increasing pressure on the pump.

$$SW_i \leq SW_{max\,i} \tag{2.8}$$

Where SW_i is represented the number of pump switching for pump groups i; $SW_{max\,i}$ is designate the maximum number of pump switching for pump group i.

2.8. Statistical comparison using SPSS

The SPSS was used to a compare the optimization results which are cost, pressure, velocity, and flow. Pre-optimization and post-optimization of the WDS were used in both sets of results due to non-normal distributions, small sample sizes, and high variance within groups, non-parametric tests were selected to analyze the data from the different type of statistical tests. A Wilcoxon signed rank test is the greatest method to achieved for non-parametric tests (Bello et al., 2015) to determine whether there was an increase or decrease the cost, velocity, pressure, and flows in optimization across all contributors from pre-optimization to post optimization results.

3.   Results and discussion

3.1. Evaluation of existing water distribution system

3.1.1.   Water balance

The existing water balance evaluation in the study area was quantified by computing the difference between the values of water production and consumption data. The total water loss in the town which accounts 13.9% of the total water production. Nevertheless, the Wukro town results indicate the existing WDS showed that a low amount of total water loss is detected in the WDS due to the water supply project is constructed in the recent year (2016). However, the water supply projects are expected that the water loss is minimum at the beginning of the design period and increase gradually with the time in the expected service life of the system unless immediate leakage detection.

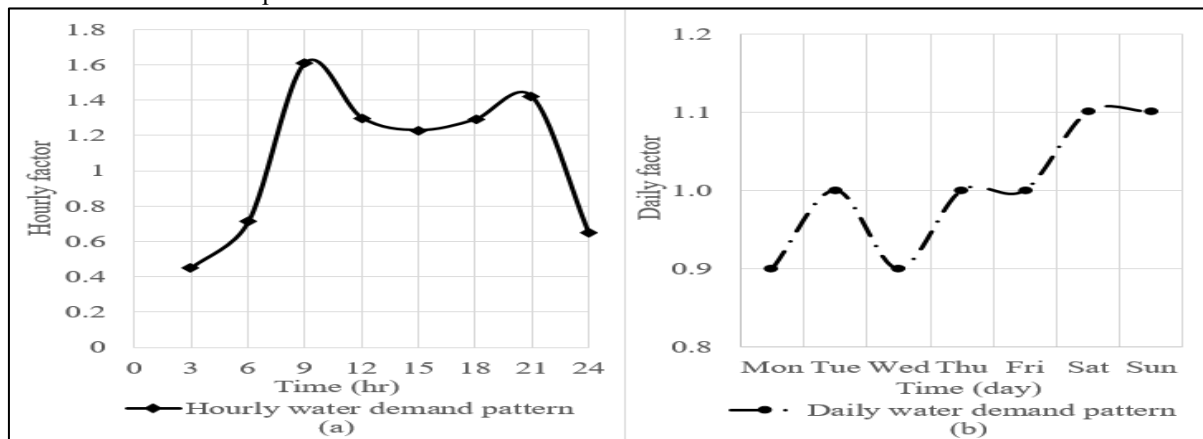### 3.1.2.   Water demand pattern

Figure 3.1 Water demand pattern of Wukro town (hourly pattern (a) and daily pattern (b))

The results have been observed that as a town the high hourly water consumption (peak hour demand) is observed early in the morning (6:00 AM to 9:00 AM) which is 1.61 factor (Fig. 3.1a), customers of Wukro town wake to begin their daily routine to use for residential, commercial, institutional, public and industrial activates. Whereas, the low hourly water consumption is also detected at midnight (12:00 PM to 3:00 AM) which is 0.45 factor because there is no more activates (washing clothes and residential activates with connected to a domestic water source because most of the peoples of Wukro town have been sleeping).

In case of daily patterns (Fig. 3.1b), the maximum daily demand has been found at Saturday and Sunday

(weekend) because most of the consumers have present in their home than other days used for a different purpose such as washing clothes, washing vehicles, and water recreation facilities. Besides, the minimum daily demand existed on Wednesday and Monday in a week since peoples of Wukro town have left from their home for different works in than other days.

3.1.3. Calibration results

Figure 3.2 showed that the WaterGEMS model has a good capability to predict the pressure at the node as confirmed with a higher coefficient value (R^2= 0.95). The liner regression relationship of pressure which showed a typical R^2 >0.5 is considered acceptable of model performance as per AWWA (2012). The model calibration result showed that the WaterGEMS model is a good predictor of pressure in the study area.
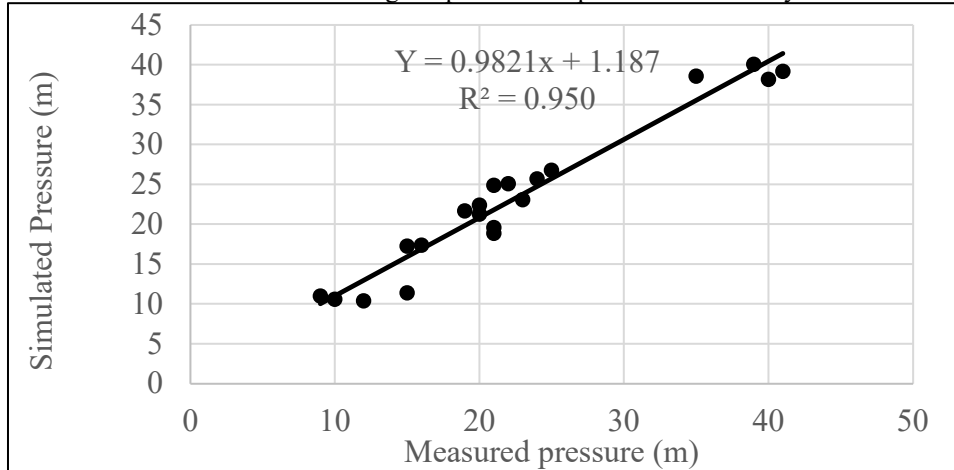

$$Y = 0.9821x + 1.187$$
$$R^2 = 0.950$$

Figure 3.2 Coefficient of determination (R^2) between measured and simulated pressure

Table 3.2 Summary of calibrated performance criteria

Performance criteria	Calibration
ME	-0.79
RSME	2.23
R^2	0.95

The performance evaluation results revealed that the WaterGEMS has a promising approach to simulate the water pressure at nodes in the WDS. According to ATSDR (2000) explained that an ME of pressure difference of ± 15.2kPa (± 1.52m) with a maximum difference of ± 50.3kPa (± 5.03m) characterizes a good performance set. However, the ME of the study area as shown in Table 3.2 (ME= -0.79) and the maximum difference is -3.9m. Therefore, these results estimated pressures indicate that the WaterGEMS model is a very good performance of pressure in the study area.

3.1.4 Hydraulic model results

The hydraulic analysis of the existing WDS has been carried out to evaluate the hydraulic behaviors by considering the pressure and velocity in existing WDS. The skeletonized pipe diameter within pressure and velocity results of Wukro town includes 182 pipes of different materials, 135 junctions, 8 pumps, 7 water source reservoir, and 7 tanks as shown in Figure 3.3 and 3.4. After the model was calibrated, the current water production in the existing WDS was evaluated under consideration of the required water patterns in the study area.
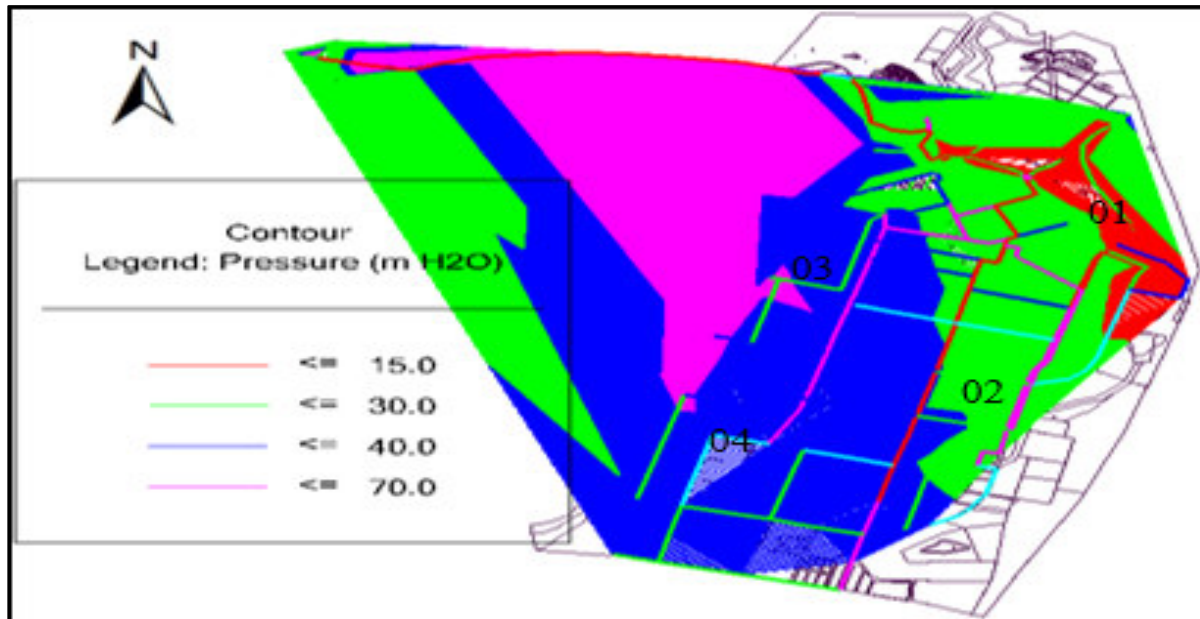
Figure 3.1 Map of pressure existing distribution system at Peak hour demand

Note: 01, 02, 03, and 04 are Kebeles (local village name) which are located in the study area.

The hydraulic model result for pressure at the PHD in the existing WDS is illustrated in Table 3.3, which compares the allowable, minimum, and maximum pressures in the nodes.

Table 3.3 Hydraulic results of pressures

Pressure (m)	Number of Nodes	Percentage (%)
<15	21	15.6
15-30	60	44.4
30-40	45	33.3
40-70	7	5.2
>70	2	1.5
Total	135	100

It has been observed the 112 nodes (82.9%) in the existing WDS satisfied the recommended pressure from 15m to 70m of water as per Ethiopian standard (MoWR, 2006). Some nodes were below the recommended values which included 21 nodes (15.6%); whereas a very few nodes were above the recommended values which hold 2 nodes (1.5%). It is, therefore the whole parts of the study area particularly the minimum pressure nodes may not get sufficient water at Peak Hour Demand (PHD) time (9:00 AM) in 01 Kebele of the town. On the other hands, the maximum pressure at nodes may also cause very high water leakage and thus lead to an increase in the high cost for maintenance and energy consumption in Abraha Weatsibeha of water sources.

The hydraulic model result for velocity at the PHD in the existing WDS is presented in Table 3.4, which compares the allowable, minimum, and maximum velocities in the pipes.

Table 3.3 Hydraulic results of velocity

Velocity (m/s)	Number of Pipes	Percentage (%)
<0.3	112	61.5
0.3-0.6	36	19.8
0.6-1	20	11.0
1.0-2.0	14	7.7
2.0-3.0	0	0
Total	182	100

The model results are displayed that 38.5% (70 pipes) in the existing WDS met the required design velocity in Ethiopian standard (0.3-3 m/s) (MoWR, 2006). However, 61.5% (112 pipes) were found below the minimum velocity standards and thus may cause low water quality due to increase in the age of water in the pipes line. Practically, the deterioration of the pipe links has been experienced insufficiency of flow throughout the all Kebele 01 to 03 in Wukro regarding the minimum velocity of the WDS.
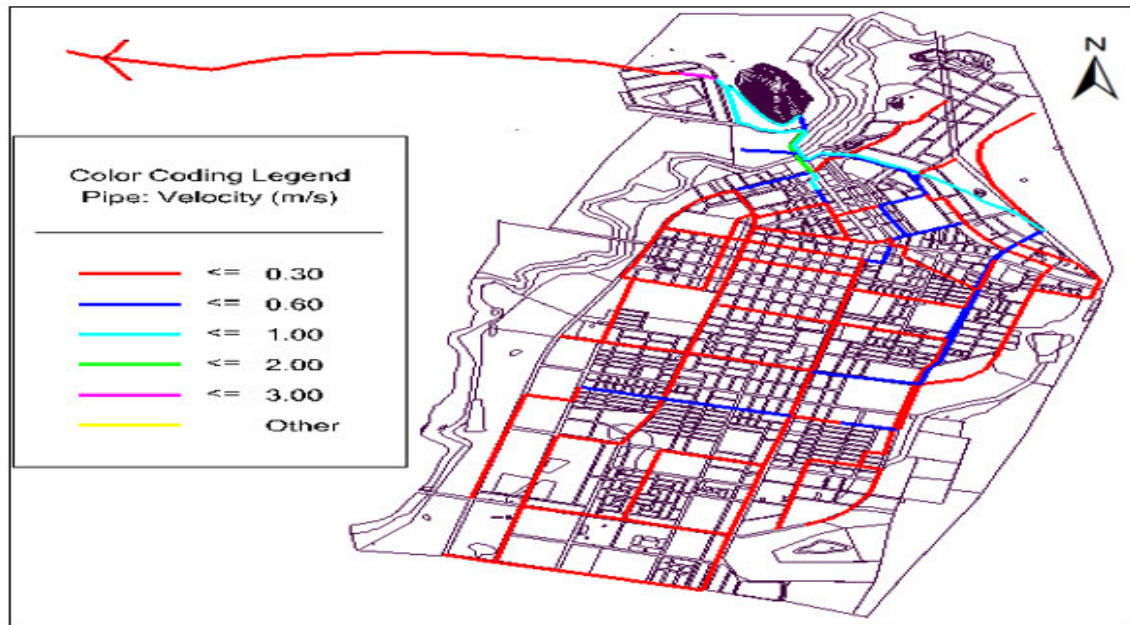
Figure 3.2 Map of velocity existing distribution system at PHD

WaterGEMS results revealed that the designed pressure and velocity did not meet the whole network of the existing WDS in the study area. As a result of this, the required water demand did not deliver to the end users in the study area. The flow in the pipe has been experienced inadequacy of flow and siltation problem throughout the link depend on low velocity. This is also exacerbated the pipe to overstated leakage value and pipe burst which in turn bring a shortage of delivered water, which is the reason for lack of water supply in the town. Those problems are leads to optimize the pipe diameter and energy consumption.

3.2. Optimized pipe diameter

The optimization of the pipe diameter of the WDS was carried out based on commercially available pipe sizes (Section 3.3.4). The optimized pipe diameter skeleton in the study area is illustrated in Figure 3.5, which pipe optimization is undertaken during the PHD (9:00 AM) at the nodes due to the reason once satisfied the required demands in the distribution systems means then the other water demands are fulfilled (Ali *et al.*, 2015).

All the comparison of the before and after optimization of hydraulic parameters was performed using the nonparametric test (the populations are not normally distributed) based on Wilcoxon signed rank test as the results of test summary $P<0.05=\alpha$ level of significance of the cost, pressure, velocity and flow. In the case of the significance of the optimization of statistical analysis, the results are explained according to the desired parameters and importance are discussed in the subsequent sections.
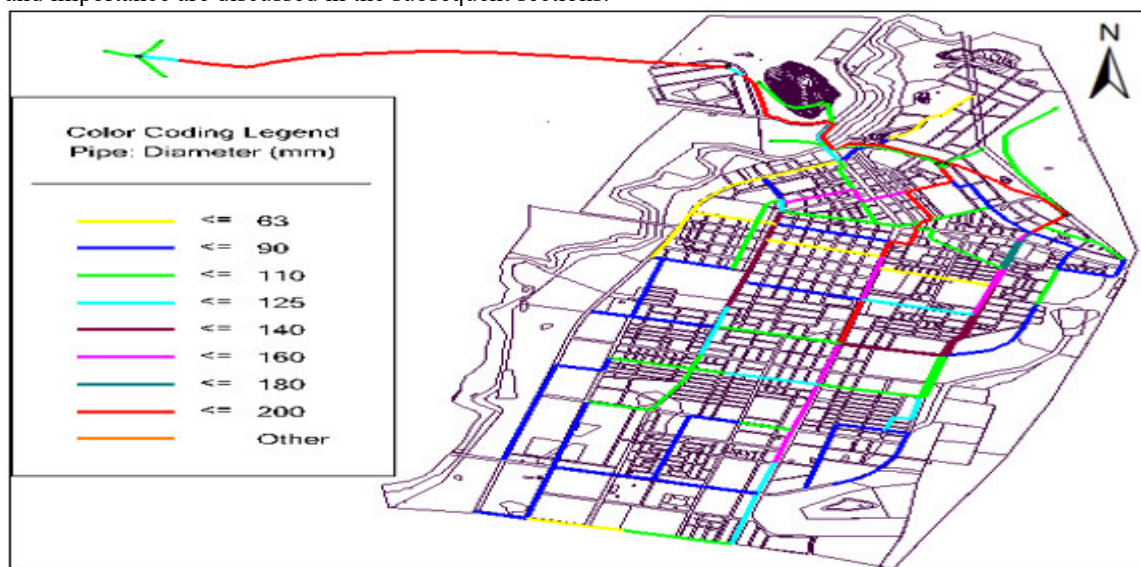

Figure 3.3 Optimized pipe diameter of the WDS in the study area

3.2.1. Cost of pipe diameter

The cost comparison of the pipe diameters was performed before and after optimization using a nonparametric test (the populations are not normally distributed depend on Wilcoxon signed rank test type). The computed test results for the pipe diameter costs are given in Tables 3.5.

Table 3.5 Wilcoxon signed rank test for cost of pipe diameter

	Ranks	N	Mean Rank	Sum of Ranks
Pre-cost - Post-cost	Negative Ranks	13[a]	30.38	395.00
	Positive Ranks	73[b]	45.84	3346.00
	Ties	31[c]		
	Total	117		

a. Pre-cost < Post-cost
b. Pre-cost > Post-cost
c. Pre-cost = Post-cost

The Wilcoxon signed rank test for the cost of pipe diameter showed that majority of pipes have positive ranks with sum ranks about 3346. Hence, positive means that the costs of pipes were higher before optimization than after optimization. However, some part of the pipes showed negative rank which has a sum ranks of 395, those results have described that the costs of the pipes were higher after optimization than before optimization. Ties pipes (31 links) showed the number of pipes sized did not change before and after optimization as analyzed the Wilcoxon signed rank test values.

In the case of direct comparison, the pipe optimization results revealed the initial cost of the pipes was reduced about 27,448,691.4 ETB from the initial cost of the pipe about 30,362,878.7 ETB. It thus optimization of the WDS by using WaterGEMS is to minimize the total cost of the pipe to be invested about 9.6% of the total cost of the pipes by commonly used hydraulic models software such as EPANET.

3.2.2. Pressure

A statistical comparison of the pressures of nodes pre and post-optimization using Wilcoxon signed rank test type was computed and its results are presented in Table 3.6.

Table 3.6 Wilcoxon signed rank test for pressure at nodes

	Ranks	N	Mean rank	Sum of ranks
Pressure After - Pressure Before	Negative Ranks	15[a]	25.53	383.00
	Positive Ranks	84[b]	54.37	4567.00
	Ties	0[c]		
	Total	99		

a. Pressure After < Pressure Before
b. Pressure After > Pressure Before
c. Pressure After = Pressure Before

As observed Table 3.6 most of pipes can provide sufficient pressure at the node as compare in Wilcoxon signed rank test confirmed by a positive rank with the sum of ranks about 4567.00. It thus the optimized pressures are increasing the pressures post-optimization than before optimization because positive rank has much greater sum than the negatives ones and zero Tie shows that the two pressures before and after optimization compare are not equivalent in any way. In fact, some junctions have less pressure in the optimization pipe compared to the existing pressure at nodes. However, all pips in the optimization networks were fulfilled the required pressure as per Ethiopian design standard. This shows that the pressure of the post-optimization pressure of nodes is higher than that of the pre-optimization due to the decrease the pipe diameter size

The WaterGEMS model result outputs for pressure before and after optimization within the 24 hours. The pressure at nodes showed that the values of minimum pressures in the existing nodes were increased from 7.9m to 16.0m at J-15, as 8m to 15.6m at J-23, and 8m to 15.3m at J-24 after pipe optimization. In a similar fashion, the optimized network is also improved the maximum pressure at nodes in the existing WDS from 31.1m to 38.1m at J-55, 29.2m to 34.7m at J-60, and at 30.8m to 35.6m at J-64. This shows that the pressure of the post-optimization of nodes is higher than that of the pre-optimization due to the decrease and increase of the pipe diameter sizes. After all, the minimum pressures occur at the same nodes but maximum pressure shifted to another node.
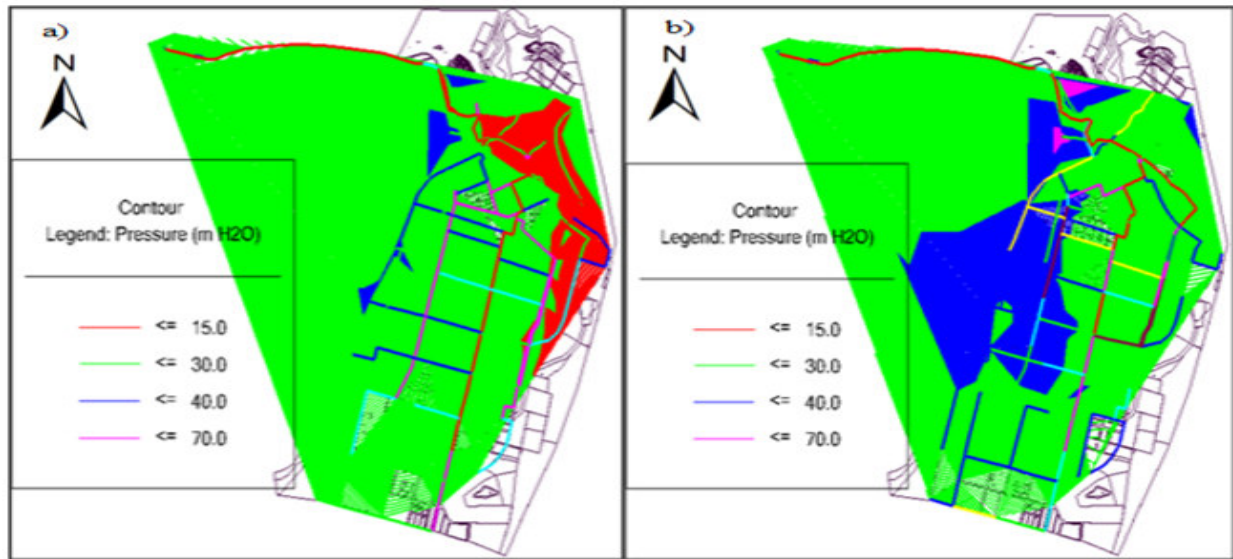
Figure 3.6 Pressure of nodes at PHD (before optimization (a) and after optimization (b))

According to the Ethiopian standard MoWR (2006) the pressure at any hour demand must be 15m to 70m of water. Unfortunately, an analysis revealed that the pressure in the pipes in Wukro town is too low (Fig. 3.6a). In such a way the minimum and maximum pressures are 15.3m and 38.1m (Fig. 3.6b) of water respectively achieved after optimization to become sufficient pressures at all nodes. After optimization, the pressure is sufficient flow to customer water demand and the problem of shortage of supply is resolved.

3.2.3. Velocity

The statistical rank test results for velocity at pipes in Table 3.7 shows that the sum of the positive rank is higher than that of the negative rank, it means that the velocity before optimization is lower than the velocity after optimization in the pipe networks. Zero "Ties" it means that the velocity before and after optimization is no equivalent in pipe networks. This shows that as the diameter size decrease the velocity of flow increase.

Table 3.7 Wilcoxon signed rank test for velocity at pipes

	Ranks	N	Mean Rank	Sum of Ranks
	Negative Ranks	25[a]	49.94	1248.50
After Velocity	-Positive Ranks	92[b]	61.46	5654.50
Before Velocity	Ties	0[c]		
	Total	117		

a. After Velocity < Before Velocity
b. After Velocity > Before Velocity
c. After Velocity = Before Velocity

There was a slight increase in velocity after optimization by considering the statistical results, Nevertheless, the direct comparison it is observed that the maximum velocity is at links P-13 (1.43m/s) and P-31 (1.47m/s) compare to the similar pipe before optimization which was 1.23m/s and 1.04m/s respectively, meaning that the velocity increase because of the diameter decrease 200mm to 150mm and 200mm to 180mm sizes respectively. Again after optimization, the minimum velocity at links P-59 and P-38 is 0.34m/s and 0.37m/s respectively, while before optimization it was 0.1m/s and 0.12m/s respectively while its diameter of the pipes is shifted from 110mm to 75mm. This displays that the velocity of the post-optimization of pipes is higher than that of the pre-optimization of the WDNs.
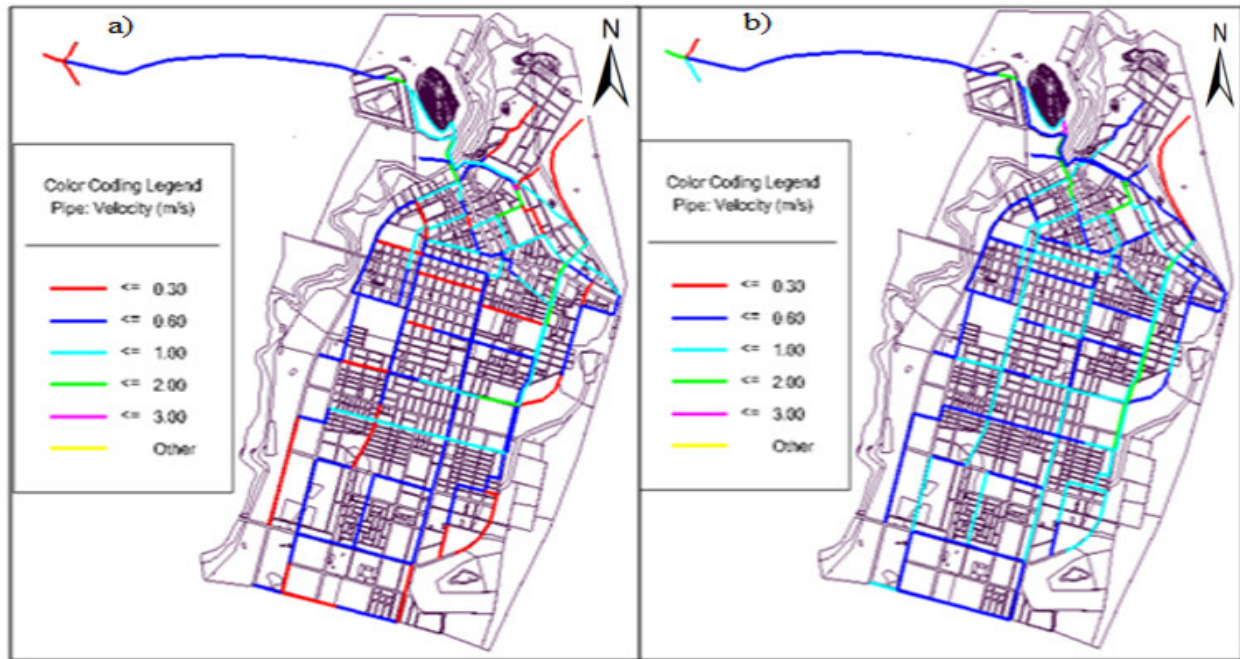
Figure 3.7 Velocity of pipes at PHD (before optimization (a) and after optimization (b))

It detected that the main problem of the system is not only the low pressure at particular locations of PHD but also the low velocity in the pipes around almost the whole system. The minimum flow velocity in the pipes should not be less than 0.3m/s to avoid poor water quality based on the Ethiopian standard. Unfortunately, an analysis revealed that the velocity in the pipes within Wukro town is too low, where 0.1m/s (Fig. 3.7a) is the average velocity.

According to MoWR (2006) the velocity must be 0.3m/s - 3m/s of the flow. As illustrated in Figure 3.7b, the minimum and maximum velocity is 0.34m/s and 1.60m/s respectively, achieved after optimization is sufficient flows of water to the consumer. This solution is produced satisfies performance both in terms of minimum and maximum velocity as well as in terms of 24-hour reservoir balancing.

3.2.4.   Flow

The Wilcoxon signed rank test of the flows in the pipes before and after optimization at the PHD (9:00 AM) is demonstrated in Table 3.8 The result showed that the sum of the "positive rank" is higher than that of the "negative rank", it means that the flow rate before optimization is lower than the flow rate after optimization. The "Ties" means that the flow before and after optimization is equivalent in pipe networks. This observed that the optimized pipe diameter of the pips to becomes sufficient flow in the distribution system than before optimization.

Table 3.8 Wilcoxon signed rank test for flow at pipes

Ranks		N	Mean Rank	Sum of Ranks
After Flow - Before Flow	Negative Ranks	27[a]	62.69	1692.50
	Positive Ranks	69[b]	42.95	2963.50
	Ties	21[c]		
	Total	117		

a. After Flow < Before Flow
b. After Flow > Before Flow
c. After Flow = Before Flow

Finally, after the optimization the selected diameters and the hydraulic analysis result of node and link values of the network at the end of the final run of the solver to have sufficient pressure to extract water and thus the flow in all pipes or links may adequate enough to withstand the pressures at nodes. Because of all velocity and pressure of water results are under the allowable range defined by constraints.

3.3. Optimized pump energy

The pump schedule optimization was carried out for optimized network systems in the study area using WaterGEMS in the Darwin Scheduler. Figure 3.8 illustrates the pump schedule in hydraulic step time 24 hr for both current and optimized pipe networks of Wukro town WDS. The energy consumption in the study was optimized based on the tanks level and numbers of pumps which were switched on and off in the network system.
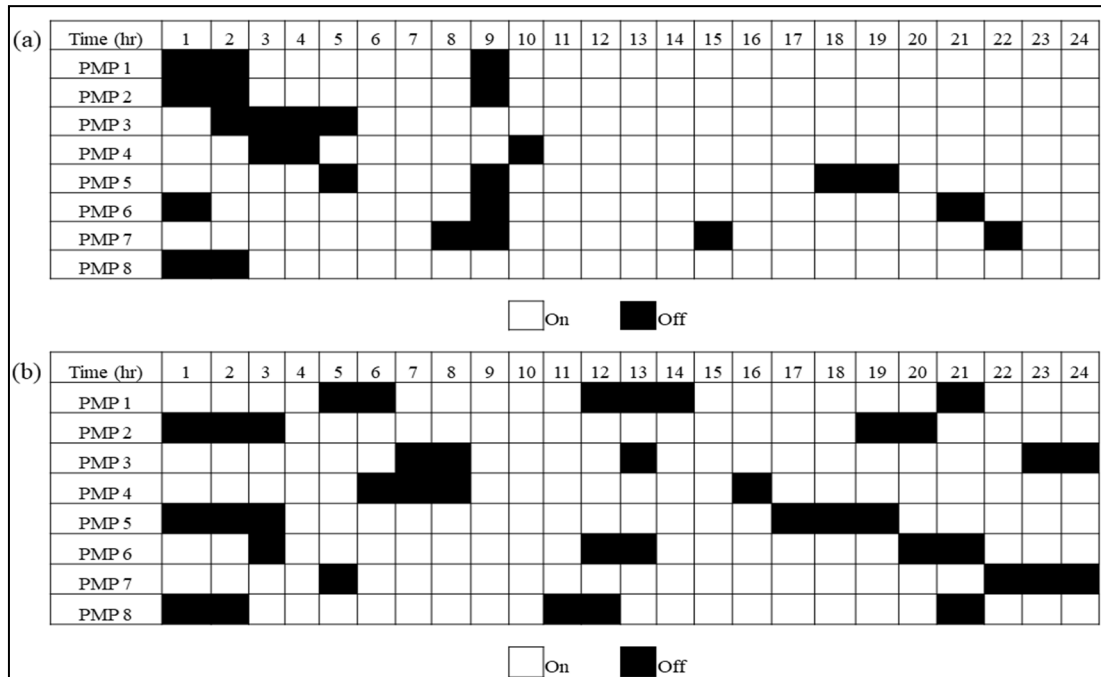
Figure 3.8 Pump scheduling of pumps (current (a) and optimized (b))

The optimized pump schedule result showed that all pumps in the optimized pumping schedules got sufficient time to rest and thus may increase the ages of the pump. At the beginning of pumping schedule (Fig. 3.8b) only the PMP1, PMP3, PMP4, PMP6, and PMP7 were working for the first average four hours while, the pumps PMP 2, PMP 5, and PMP 8 becomes inactive. In addition to this, results also demonstrated that the optimized pimping schedules avoided continuous working of pumps for long periods. The current pimping schedule (Fig. 3.8a) revealed that some pumps were working without much of break (PMP 8 for 22 hours). Besides, most of the pumps were delivered water during the morning (9:00 AM to 6:00 PM) due to this manually schedule more energy was consumed.

The pumping schedule examination results revealed that this continuous pumping operation can increase net energy consumptions and high cost to maintain the operation of the pump. A similar change occurs for optimized pumps PMP1, PMP4, PMP6, PMP7, and PMP8, which are switched off and replaced by PMP2, PMP3, and PMP5 that originally remained between switch controls. The scheduling is continued by replacing each other's until partial and full level of the tanks. As expected the consumptions of pumps that are also included in the optimization show variations after the optimization. This observes that the pumping hour decrease the energy consumption is decreases based on the tanks levels variations.

3.3.1   Energy consumption

The impact of energy consumption for water pumping is shown in Figure 3.9, which compares the daily energy request of each pump in both the current and the optimized configuration.
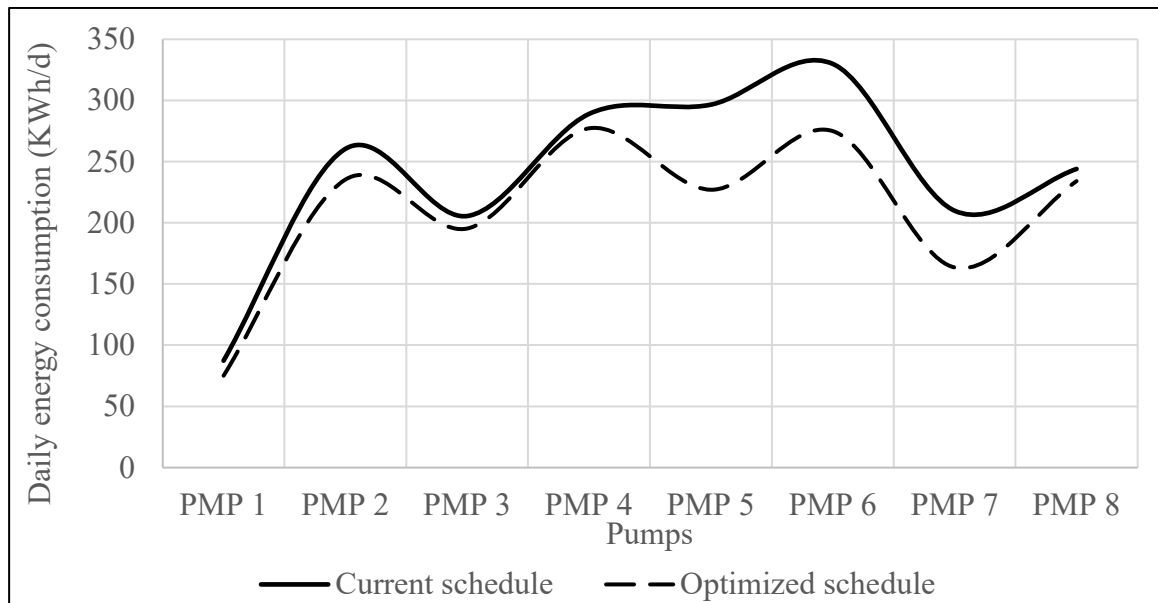
Figure 3.9 Daily energy consumptions of the current and optimized configuration

The daily energy consumption of WDS is reduced after optimization from 1922.8KWh/d to 1683KWh/d. It thus the optimized pumping schedule reduced the daily energy consumption by about 12.5% (239.8KWh/d) for the whole WDN. This result indicates that in pumping water during the minimum demand hours (during the night) when the energy cost is minimal in order to fill tanks, and occupancy tanks feed the WDN during the day. This problem formulation leads to the optimizer attempting to reduce both energy usage and leakage. The other studies Ali *et al.* (2015) have the daily pump energy consumption is reduced to 676.6 KWh depend on the scheduled time.

3.3.2.   Cost of energy

The total operational cost of before and after optimization of the pumping system of the monthly and yearly is shown in Table 3.9.

The cost reduction of energy consumption in the study area was applied in water supply pumps from the borehole to tanks and beside from collection chamber to the tanks.

Table 3.9 Before and after optimization of energy cost (daily, monthly and annually)

Label	Current energy consumption cost			Optimized energy consumption cost		
	Daily Energy (KWh/d)	Monthly costs (ETB)	Annual cost (ETB)	Daily Energy (KWh/d)	Monthly costs (ETB)	Annual cost (ETB)
PMP 1	87.5	1843.6	22122.9	75.1	1581.7	18980.5
PMP 2	260.4	5494.9	65939.2	235.5	4969.1	59629.1
PMP 3	205.7	4339.8	52077.2	195.4	4122.2	49466.9
PMP 4	288.9	6096.8	73161.7	277.3	5851.8	70222.0
PMP 5	296.7	6261.5	75138.4	227.1	4791.7	57500.3
PMP 6	329.7	6958.4	83501.2	275	5803.3	69639.1
PMP 7	209.9	4428.5	53141.5	163.4	3446.5	41357.5
PMP 8	244.0	5148.6	61783.1	234.2	4941.6	59299.6
Total	1922.8	40572.1	486865.2	1683	35507.9	426095.1

As shown the Table 3.9 the model predicts decrease in the daily energy consumption of 239.80KWh/d for the whole WDN. A comparison has been made between the cost of the current network operation and the optimized operation. Based on the current schedule hours the annual cost was 486,865.2 ETB/year whereas after optimized the schedule of time decreased to 426,095.1 ETB/year. The reduction in the energy consumption results in an expected saving of 60,770.1 ETB/year (12.5%) and justifies the use of WaterGEMS (Darwin Scheduler) for this type of problem, and indicates that other manually schedule designs schemes may be costing far more than necessary.

4.   Conclusions and Recomnadations

In this study the WaterGEMS model has been introduced and carried out to the least cost design of the water distribution networks in the case of Wukro town. The results showed that the least cost solutions were obtained which fulfilled the required flow and pressure consistently at the node. The best results accomplished revealing savings of 9.6% and 12.5% based on the cost of the pipes and energy consumption respectively in the existing network also confirmed that the WaterGEMS optimization technique has constantly obtained lower cost solutions

used by many design engineers.

The findings of this study showed that the WaterGEMS pumping operation was an attractive approach for efficient scheduling of the water pumping operations to accomplish management targets namely the minimum cost, quality upgrading, and pressure regulation. The optimized water distribution networks in the study area was controlled the water pressure within the network nodes and it thus may reduce the water losses in the system is noteworthy as the more complex problems to deal with concerning water supply networks.

The results are statistically significant difference of pre-optimization and post-optimization such as cost, pressure, velocity, and flow. The findings of this study can be concluded that WaterGEMS modeling approach based on genetic algorithm is a promising alternative for the optimal design of water distribution systems, as they demonstrated for the Wukro town water distribution case study considered in terms of hydraulic pressure and computational efficiency.

In the e design of water distribution networks in urban centers of Ethiopia and other country, application of WaterGEMS models can be used effective tool for least cost design of WDS. Before applying the optimized pipe diameter and pump scheduling in the case of new WDS, further research should be done regarding the cost of construction and maintenance.

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