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Research Paper

Sustainable Clean Water Distribution System
[Case Study in Landungsari Village, Malang Regency East Java]

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ABSTRACT: *As one of the primary natural resources in the universe, water is a fundamental element that must be sufficiently available for living beings. As such a basic element, the presence of water is highly anticipated throughout life. Therefore, the availability of water in life is absolute and non-negotiable. In its utilization, it is essential to employ wise methods to ensure its sustainability and preservation. Similarly, in the study location, the existing potential is utilized to meet daily clean water needs. However, these needs have not been fully met due to the uneven distribution of water resources. This is because the geohydrological conditions of shallow groundwater are uneven, with some areas located in shallow soil layers and others in deeper soil layers, making them inaccessible for direct use by the community. Another issue is the suboptimal mapping and arrangement of the water network. This study aims to enable a more optimal planning of the network layout as expected. BUMDes, as the village government's representative handling this matter, is responsible for enhancing water resilience for the community, particularly regarding clean water. Through this study, BUMDes "Tirto" strives to initiate breakthroughs in planning an effective and efficient network arrangement to improve clean water services for the community. Based on previous studies regarding the groundwater potential at SB.1, the estimated potential is 8.74 lt/s, and at SB.2, it is 7.11 lt/s. Referring to data from the last 10 years and projections up to 2032, the population is estimated to reach 9,574 people. The projected clean water demand by 2032 averages 7.82 lt/s, with a daily maximum of 9.38 lt/s and a peak hour demand of 13.28 lt/s. The groundwater potential at SB.1 and SB.2, based on earlier studies, still meets the demand, even up to the confined zone depth. This indicates that the existing potential can still fulfill the community's needs by 2032. To ensure the sustainability of this potential, its utilization must not be excessive. The analysis results, based on network optimization, recommend the use of pipes with diameters of 4", 2 ½", 2", 1 ½", 1 ¼", 1", ¾", and ½" for SB.1, and diameters of 4", 2 ½", 2", ¾", and ½" for SB.2. The analysis also recommends the use of HDPE (High-Density Polyethylene) SDR17 PN10 pipes, considering the available pressure levels. The planned capacity of the required reservoir is 95 m³ at SB.1 and 78 m³ at SB.2. To ensure sustainability, the utilization of groundwater potential should not exceed optimal capacity, and environmental conservation should be implemented by infiltrating rainwater into the ground.*

KEYWORDS: *Distribution; Clean water; Capacity; Network*

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I. INTRODUCTION

Climate change is a global issue that is occurring all over the world. The decline in environmental quality, such as environmental damage, deforestation, and other factors, is causing disasters everywhere. Floods during the rainy season, droughts, and water crises during the dry season are becoming increasingly common[1]. Extreme climate conditions are causing various problems to arise. Environmental damage is also one of the causes of these extreme climate changes. Environmental degradation leads to shifts in climate and seasons, resulting in frequent occurrences of droughts and floods everywhere[2].

Landungsari Village is the study area located in Dau District, Malang Regency, and is situated between the western part of Malang City and the eastern part of Batu City. The study area lies on the southern side of the Brantas River, which divides Greater Malang, originating from the upstream Sumber Brantas in Batu City. It directly borders Tlogomas Village to the north and east, Merjosari Village in Malang City to the south, and Mulyoagung Village in Malang Regency to the west[3].

The community in the study area primarily works as farmers, with some being agricultural laborers or employed in other sectors related to agriculture. The agricultural potential in the area is significant, especially in the vegetable and fruit farming sectors. The main agricultural commodities include chili, tomatoes, carrots, and oranges. Some residents also manage agricultural tourism, particularly orange-picking tourism, which is commonly found in the study area. Over time, infrastructure in the study area has continued to improve, including schools, healthcare facilities, and markets, as well as social and cultural activities such as traditional dance and customary events[4].

The potential and availability of groundwater for clean water in the study area are uneven. The northern part has potential for shallow groundwater, located at an elevation of approximately 15 meters below the ground surface. Meanwhile, the central and southern parts tend to have deeper groundwater, around 20 meters or even more than 40 meters below the ground surface. This condition requires specific handling to utilize its potential effectively, using technology to meet the clean water needs of the surrounding community[5] [6]. In addition, detailed mapping of the area is necessary to determine the optimal service zones based on previous findings regarding the potential and depth of the aquifer layers[7][8][9][10][11][12].

The population in the study area is classified as dense and is considered highly strategic due to its proximity to both Malang City and Batu City. According to data from the Central Statistics Agency, the current population is approximately 9,330 people and continues to grow. With such a dense population, the need for clean water is one of the top priorities to support the daily activities of the community. This situation requires adequate infrastructure and an optimal distribution network system to ensure convenience for the residents and directly improve public health standards.

The raw water sources in the study area have not been utilized optimally due to the inadequate clean water supply system. The demand for clean water in the study area has not been met proportionally with the available supply[13]. This condition has broad impacts on the acceleration of both economic and social development. To meet the demand for clean water, it is necessary to increase the availability capacity by exploring potential resources based on previous studies. As the population grows and socio-economic dynamics evolve, the need for clean water also increases, requiring easier access to clean water.

The current climate change significantly affects environmental conditions, including the vulnerability of clean water availability for communities and the potential groundwater resources that can be utilized. To prevent the decline of groundwater potential and ensure its sustainability, active community participation in environmental conservation, particularly in recharge areas, is essential[14][15]. The current condition in the study area still relies on makeshift networks that have not been systematically planned. Therefore, improvements, well-structured and optimal planning, as well as capacity testing, are necessary.

Therefore, to achieve good access to clean water services, professional handling is required, starting from the water source, network, treatment, and distribution, and up to the service provided to the community. To provide a more directed overview for rural communities, the development of clean water infrastructure and facilities necessitates a clean water supply plan aligned with a long-term (10-year) plan. This plan serves as the initial stage of the clean water supply system planning for an area, encompassing periods, stages, projections, system components, cost estimates, and expected benefits, following the Ministry of Public Works and Public Housing Regulation on the Implementation of Drinking Water Supply System Development[16].

II. METHOD

The method applied in addressing the issue of clean water supply in the study area, particularly in regions that are not fully served, involves a series of steps to ensure efficient, safe, and sustainable clean water provision. The stages required for optimizing the clean water supply system in the study area include the preparation phase, preliminary survey phase, data analysis, technical planning, and final reporting.

The preparation phase is the initial stage that must be carried out and includes preparing personnel, coordinating with relevant institutions, gathering necessary equipment, and drafting a framework. The preliminary survey phase begins with investigations and coordination with related agencies regarding topographic maps, hydrological maps, population data, and public facilities. This is followed by conducting a survey of the existing network conditions in the study area and mapping the topography of the study region.

In the data analysis phase, the objectives include determining the projected population growth for the target year, estimating the clean water demand for the target year in each service zone, analyzing the available raw water potential, and examining the topographic conditions related to the elevation of the service area. This information is then used to position the planned network for hydraulic analysis simulations, which help determine the pressure levels, dimensions of each network component, and the required flow rate that can be delivered.

In the technical planning phase, once the clean water demand for each zone has been determined and the elevations of the upstream and downstream service areas have been established based on topographic data, the next step is to optimize the network. This involves determining the required pipe dimensions and the flow rate to achieve optimal results. An optimal network is achieved when the pipe dimensions derived from the simulation are the smallest possible, the available pressure at each node meets the required standards[16] and the flow rate at each node aligns with the planned demand.

Partially, pump planning can be conducted during network and flow rate optimization, but it must also be integrated with the planned reservoir capacity. This is crucial as it relates to the efficiency of the pump's service life and the effectiveness of the reservoir capacity. Factors such as how long the pump operates and how long it rests are significantly influenced by the peak or maximum water demand, which in turn affects the required reservoir size to store water during pump operation. These considerations ultimately determine the budget needed to realize the clean water supply system.

1. Population Data

In every planning and design process across various fields, information about population size, distribution, and age composition is essential. Population projections are conducted to understand annual population growth, aiming to estimate future water demand. The accuracy or precision of population projections heavily depends on the accuracy of assumptions regarding trends in population change components. Assumptions about future birth rates, mortality rates, and migration are determined by analyzing past and present trends. Factors influencing these trends include social and economic developments, achievements in health programs, family planning, and others.

Population projections can be estimated using arithmetic, geometric, or exponential methods. The arithmetic method is also known as the average loss method. This method is used when population growth occurs periodically and remains relatively constant. The arithmetic population projection assumes that the future population will increase by the same amount each year. The geometric method assumes growth occurs geometrically based on compound interest calculations. Meanwhile, the exponential method refers to gradual increases over the years, assuming that growth occurs at a single moment within a specific period.

To determine the most relevant results among the three methods used, the obtained results must first be tested for projection method suitability by measuring the standard deviation (S) and correlation value (r). The smaller the deviation value, the more uniform or homogeneous the tested data can be concluded. Meanwhile, if the correlation value approaches 1, it indicates that the tested data is highly identical to the actual values. The equation used for the standard deviation test is as follows.

$$S = \sqrt{\frac{\sum_{i=1}^{n}(xi - x)^2}{n-1}} \rightarrow (1)$$

With **S** as the standard deviation, **xi** as the variance value (population projection), **X** as the average value, and **n** as the number of tested data points. Meanwhile, to calculate correlation, the following equation is used, where **r** is the correlation coefficient, **X** is the actual population, and **Y** is the projected population.

$$r = \frac{n\sum XY - \sum X \sum Y}{\sqrt{\left(nX^2 - \left(\sum X\right)^2\right)\left(nY^2 - \left(\sum Y\right)^2\right)}} \rightarrow (2)$$

2. Raw Water Demand

The total water demand is the overall amount of water required to support all daily human activities. This demand includes domestic and non-domestic water needs, irrigation water for both agriculture and fisheries, as well as water for urban flushing[17]. The total population water demand is calculated based on several types of needs, including domestic clean water for household connections and public faucets, as well as non-domestic clean water for places of worship and public faucets, which is estimated at 20% of domestic demand. Additionally, water loss is considered, along with an extra demand for a maximum daily requirement of 10% and a peak hour requirement of 50% of the clean water demand. This calculation is adjusted based on the standards of each region.

3. Topography

In planning a pipeline distribution route, it is essential to consider the elevation contour of the pipeline path. This ensures that water distribution can be efficiently delivered by utilizing gravity as a driving force. Topographic analysis can be conducted using various geospatial data, which produces contour maps of an area. These contours can be categorized into two types: those with larger intervals and those with smaller intervals. To verify the accuracy of spatial data, calibration can be performed by comparing it with direct survey elevation data using geodetic GPS.

4. Transmission and Distribution Pipes

Optimization of the raw water pipeline network is a crucial activity in planning raw water needs within a raw water supply system. There are three main requirements to consider in a raw water supply system: quality, quantity, and continuity. Water distribution must take into account the raw water supply system to ensure access reaches all service areas. Water flow is considered ideal when the movement of water maintains a constant velocity at each point in the pipeline and flows steadily due to the influence of Earth's gravity,

According to the continuity equation, the flow rate in a branching pipe remains constant despite differences in cross-sectional areas and follows the equation $\mathbf{A_1\ V_1\ = A_2\ V_2\ = A_3\ V_3}$. To simplify the solution of branching pipe problems, the iterative approach used is the Hardy-Cross method. The Hardy-Cross (HC) method is one of the techniques used to determine flow distribution in a pipe network. Through the Hardy-Cross method, pipeline network problems are solved using an iterative calculation model based on the fundamental equations of flow continuity and energy loss (head loss).

In general, energy loss in a pipeline network is categorized into major losses (due to friction) and minor losses (caused by bends, branches, fittings, cross-section changes, etc.). The total energy loss is calculated using Bernoulli’s equation, as follows.

$$Z_1 + \frac{P_1}{\gamma} + \frac{v_1{}^2}{2g} = Z_2 + \frac{P_2}{\gamma} + \frac{v_2{}^2}{2g} + hf \rightarrow (3)$$

Major energy loss is caused by friction between the fluid and the pipe walls along its length, which depends on fluid viscosity. The approach used for this is the ***Darcy-Weisbach (DW) equation*** or the ***Hazen-Williams (HW) equation***. Meanwhile, minor energy loss is calculated using the ***Triatmodjo (BT) equation*** for cross-sectional changes, branches, or curved bends, while for miter bends, the ***EMA equation*** can be used[18][19].

$$\text{Mayor of DW: } hf = f\,\frac{L}{D}\,\frac{V^2}{2g} \rightarrow (4)$$

$$\text{Mayor of HW: } hf = \frac{Q^{1.85}}{(0.2785\, D^{2.63}\, C}L \rightarrow (5)$$

$$\text{Minor of BT: } hf = K\frac{V^2}{2g} \rightarrow (6)$$

$$\text{Minor of EMA: } hf = C_{pd}\frac{V^2}{2g} \rightarrow (7)$$

$$\text{Method of HC: } \Sigma Qi = qj \rightarrow (8)$$

In the HC method, **Qi** represents the flow rate in each pipeline **i** meeting at node **j**, while **qj** represents the flow rate at each node **j** (the demand or consumption flow rate). The total energy loss in a pipeline network loop must be equal to zero, following the equation $\sum$**Ki.Qi |Qi| = 0** for all loops.

III. ANALYSIS RESULT AND DISCUSSION

The analysis and review of the study’s evaluation results identify the factors that support problem-solving, as well as the challenges and obstacles faced in addressing the issue. Additionally, the discussion provides recommendations and further steps that can be taken to enhance sustainability and the positive impact of groundwater resource development in the study area[4]. An effective network system and operational pattern also have a positive impact on the health of clean water supply infrastructure as well as the sustainability of its potential.

1. Location and Area

The study area is located in Dau District, Malang Regency, East Java. Geographically, it lies between Malang City and Batu City, at coordinates 7°21'–7°31' S and 110°10'–111°40' E. The study area covers approximately 298 hectares and is divided into three hamlets: Rambaan, Bendungan, and Klandungan. It consists of 13 neighborhood units and 50 community units. Topographically, the study area is situated at the foot of Mount Panderman, with an elevation ranging from 540 to 700 meters above sea level. Due to its elevation, the area consists of highlands and hilly terrain with varying elevations.

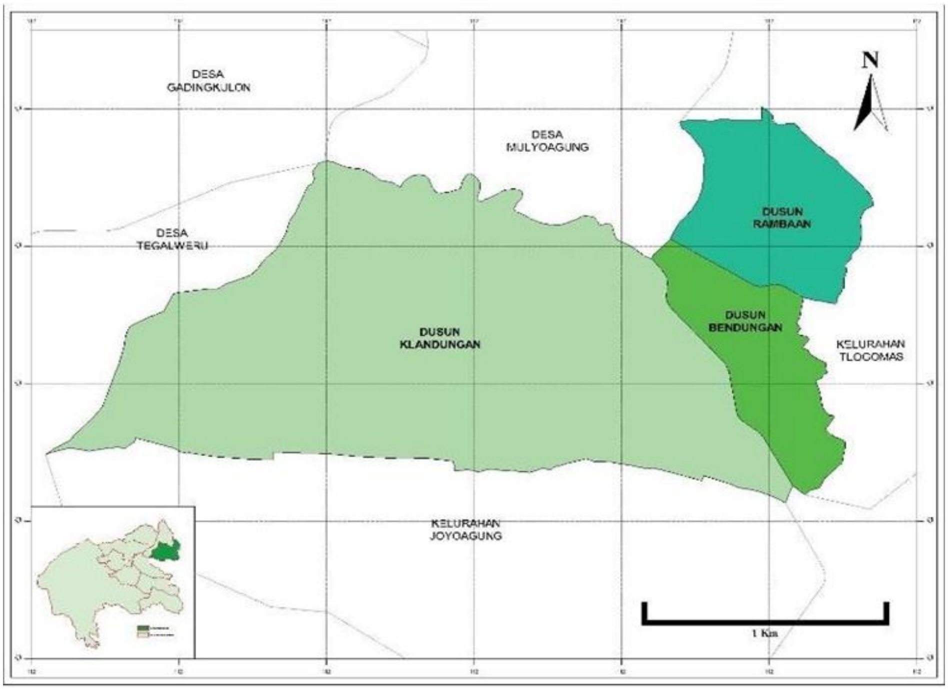


Figure 1. Map of Study Area

Based on the topographic map, the study area is part of the Brantas Watershed (WS Brantas), covering 354,395 hectares, which accounts for 25.13% of the total area of WS Brantas. The highest rainfall in 2015 occurred in October, reaching 158 mm, while the lowest rainfall was in August, at 1 mm. Year-round rainfall significantly affects river discharge and groundwater levels. According to BBWS Brantas climatology data (2009–2014), the average relative humidity (RH) showed an increasing trend, while the average temperature experienced a decline. In 2014, the highest average air temperature was recorded in January at 28.45°C, while the lowest average temperature was in September at 24.21°C, with a humidity level of 98%[20].

According to the Central Bureau of Statistics (BPS) of Malang Regency**,** in 2022, the study area had **a** population of 9,330 people**,** consisting of 4,659 males and 4,671 females[21]. Thus, the proportion of the population by gender includes 49.94% male and 50.06% female, indicating that the gender distribution of the population is relatively balanced between males and females. The public facilities in the study area are relatively comprehensive, ranging from educational institutions, hospitals, mosques, markets, terminals, etc.

The population data from the Central Statistics Agency[22] and the study area are used to calculate population growth projections and the required water demand. The population of Landungsari Village[23] in 2012 was 9,131 people, and in 2022, it was 9,330 people, distributed across all hamlets and neighborhood units (RW) in the study area. The projected population for the year 2034, over a period of 12 years, estimates the population of Landungsari Village to be 9,574 people (arithmetic method), 9,575 people (geometric method), and 9,575 people (exponential method).

Table 1. Population Data for the Last 10 Years

Year	2012	2013	2014	2015	2016	2017	2018	2019	2020	2021	2022
Population	**9131**	9674	9674	11428	11428	9997	9342	9575	10048	9155	**9330**

Source: [23]

Table 2. Population Data and 2034 Projection Results for the Study Area

Year	2012	2022	2034 Projection			Standard of deviation [Sd]			Coef. of correlation [r]		
	[Po]	[Pt]	[1]	[2]	[3]	[1]	[2]	[3]	[1]	[2]	[3]
Population	9131	9330	9574	9575	9575	73.31	73.55	73.55	0.99768	0.99767	0.99767

Source: Analysis Result; [1] Arithmetic method; [2] Geometric method; [3] Exponential method

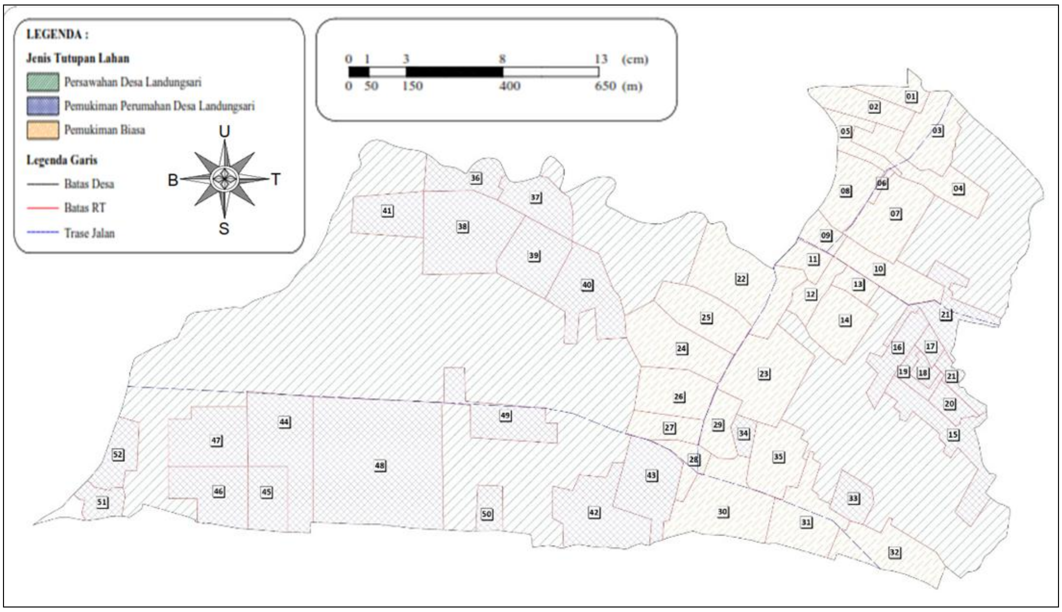


Figure 2. Block Division Map of the Study Area

2. Projection of Clean Water Demand for the Population

Based on the population projection for the year 2034, Hamlet Rambakan and Bendungan cover blocks 1 to 14 with a population of 2,738 people, Puri Landungsari Housing covers blocks 15 to 21 with a population of 1,071 people, Hamlet Klandungan covers blocks 22 to 35 with a population of 2,825 people, Oma Campus Housing covers blocks 36 to 41 with a population of 1,224 people, Bestari Housing covers blocks 42 and 43 with a population of 554 people, and the housing located above Bestari covers blocks 44 to 52 with a population of 1,163 people. The water demand for each hamlet is as shown in Table 3 below.

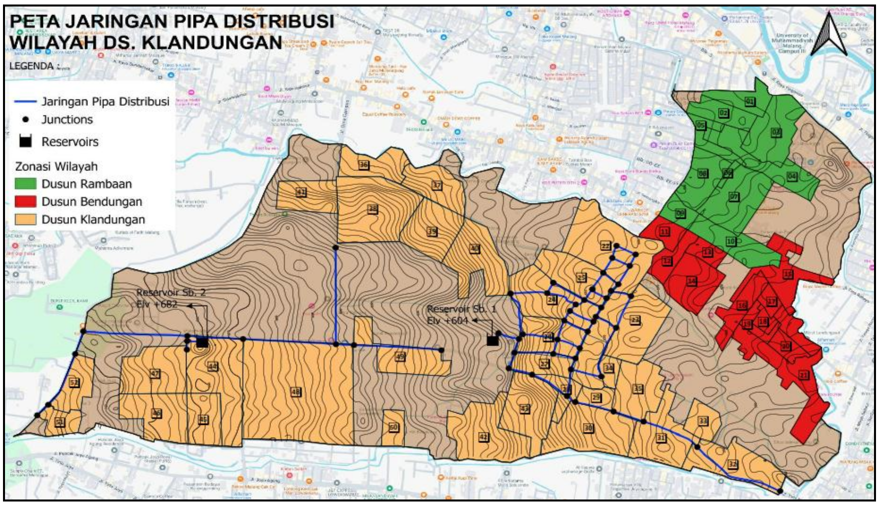
Table 3. Projection of Clean Water Demand Based on Area Description Until 2034

No	Description	Population		Daily water requirements [Q]					
				D and nD		wl	Rate	Max	Peak
		Block	People	[l/day]	[l/s]	[l/s]	[l/s]	[l/s]	[l/s]
1	Rambakan & Bendungan Village	1 – 14	2.738	160.928,95	1.86	0.37	2.24	2.68	3.80
2	Puri Landungsari Cluster	15 – 21	1.071	62.972,20	0.73	0.15	0.87	1.05	1.49
3	Klandungan Village	22 – 35	2.825	166.126,66	1.92	0.38	2.31	2.77	3.92
4	Oma Campus Cluster	36 – 41	1.224	71.968,23	0.83	0.17	1.00	1.20	1.70
5	Pondok Bestari Cluster	42 – 43	554	32.585,61	0.38	0.08	0.45	0.54	0.77
6	South side of West Clusters	44 – 51	1.162	68.369,81	0.79	0.16	0.95	1.14	1.61
Total 2034			**9.574**		**6.52**		**7.82**	**9.38**	**13.28**

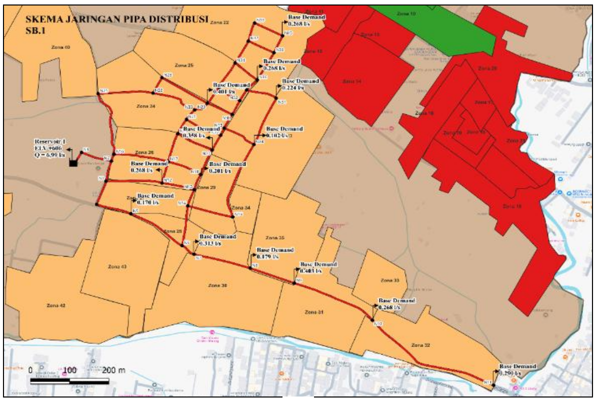
Source: Analysis Results [Domestic and non-domestic water demand with 20% water loss, 70% service coverage, assuming 30% use shallow wells independently; D = domestic, nD = non-domestic, wl = 20% water loss].

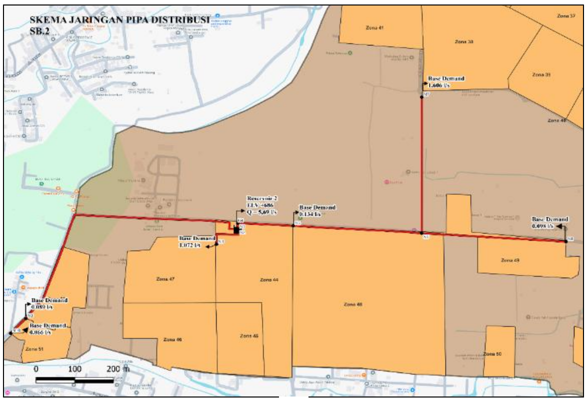
3. Network Plan

The clean water installation network is an infrastructure system designed to deliver clean water from its source to service areas, including residential buildings, commercial establishments, public facilities, and others. This infrastructure is an integral part of the clean water supply system in a region. The related elements of a professional clean water supply system include water sources, pumps, distribution pipes, storage or reservoirs, treatment systems if required, gate valves and control systems, as well as measurement and monitoring equipment. Based on previous studies in the study area, the potential groundwater availability shows a discharge potential of 8,740 l/s at SB.1 and 7,110 l/s at SB.2. The optimum discharge that can be utilized is 6,992 l/s at SB.1 and 5,668 l/s at SB.2.



(a)





(b) (c)

Figure 3. Clean Water Distribution Network Plan; (a) clean water distribution network, (b) distribution network of SB.1, (c) distribution network of SB.2.

(a) (b)

Figure 4. Reservoir and SB Location Plan; (a) SB.1 location; (b) SB.2 location

For the purpose of effectiveness and efficiency in clean water distribution services, considerations of the pump operational system and the use of reservoirs or water towers are integral parts of the operating system and heavily depend on the elevation conditions of the source and the service area. An ideal source location has a sufficiently high elevation so that the distribution system can rely on gravity for its distribution services. This way, pump operations become effective and efficient. Pump capacity and reservoir capacity are two interrelated factors. Pump capacity is determined by the service flow rate and the elevation of the service area. If a sufficiently high service source is required, it can be managed using a reservoir or water tower as the water source. The capacity of the reservoir is largely determined by the service flow rate, the pump operation pattern, and the required pressure head from the reservoir to ensure the system can operate by gravity, thereby extending the pump's service life.

The determination of the SB location refers to land ownership and the results of potential investigations based on previous studies established by BUMDes Tirto, which is always responsible for managing the provision of clean water in the study area. This is also related to the adequacy of land for operational support facilities for clean water provision, such as reservoirs and pump houses, which will be used during operation. Based on the location and elevation coordinates of the SB, the network system can then be determined. Ideally, a non-elevated storage system is used, but its elevation must cover the entire service zone so that the service utilizes a gravity system to ensure effectiveness and efficiency.

The distribution pipeline network from SB to the service zone is designed using HDPE (High-Density Polyethylene) PE100 SDR17 PN10 pipes. This choice is determined by their resistance to corrosion, flexibility, and ability to withstand pressure. HDPE pipes have a long service life and are easy to install and maintain, making them highly suitable for the water distribution network in the study area, which aligns with the contour. To facilitate maintenance, the pipeline network is planned to follow the road alignment according to the service zone.

The Distribution Pipeline Network Scheme at the SB.1 location is designed to distribute water from a reservoir with an elevation of +606, delivering a flow rate of 3,713 l/s during average hours and 6.5 l/s during peak hours to various service zones. The network consists of main and branch pipes forming a loop system, ensuring stable water flow throughout the area. Each service zone is marked by a base demand indicating water requirements in liters per second. Nodes or connection points in the pipeline network help regulate the distribution of flow and water pressure to each area.

The initial simulation process is carried out by selecting the pipe diameter based on assumptions about the system's requirements. The goal of this simulation is to control the suitability of the flow rate in each pipe segment according to the plan, as well as to determine the most effective pipe dimensions in relation to the pressure and flow rate. Hydraulic analysis is performed using the Hazen-Williams formula. The simulation results are used to calculate the most effective pipe diameter and to evaluate the performance of the distribution system, ensuring efficient flow that complies with technical standards. Referring to the standard provisions[16], the calculation of the distribution pipe dimensions is determined based on the flow during peak hours, as in the calculation of the dimensions for the Reservoir (R1) – N1 (Pipe P1) as follows:

Data:

Distance (L)	= 32.2 m	Pipe Coeff. (C)	= 130
Start Elev.	= 608 m	V min	= 0,3 m/s
End Elev.	= 602 m	V max	= 4,5 m/s
Discharge (Q)	= 6,5 l/s = 0.0065 m³/s		

Pipe Diameter:

The selection of pipe diameter is done by first calculating the minimum and maximum pipe diameters. The diameter of the pipe to be used must fall within the range between the minimum and maximum pipe diameters (Dmin ≤ D ≤ Dmax). To determine the minimum and maximum diameters, follow the steps below.

$$\mathrm{Dmax} = \sqrt{\frac{4 \times Q}{\pi \times V}} = \sqrt{\frac{4 \times 0{,}0065}{3{,}14 \times 0{,}3}} = 0{,}1661 \text{ m} = 166{,}1 \text{ mm}$$

$$\mathrm{Dmin} = \sqrt{\frac{4 \times Q}{\pi \times V}} = \sqrt{\frac{4 \times 0{,}0065}{3{,}14 \times 4{,}5}} = 0{,}0429 \text{ m} = 42{,}9 \text{ mm}$$

Pipe cross-sectional area:

Based on D-min and D-max, a 4-inch pipe diameter is used (ID 96.8mm, OD 110 mm)

$$A = \frac{1}{4} \times \pi \times D^2 = \frac{1}{4} \times 3{,}14 \times 0{,}0968^2 = 0.0074 \text{ m}^2$$

Head Loss (Hf):

$$Hf = \frac{10{,}675 \times L \times Q^{1{,}852}}{C^{1{,}852} \times D^{4{,}87}} = \frac{10{,}675 \times 32{,}2 \times 0{,}0065^{1{,}852}}{130^{1{,}852} \times 0{,}0968^{4{,}87}} = 0.323 = \frac{0{,}323}{0{,}032} = 10.036 \text{ m/km}$$

Flow Velocity (V):

$$V = \frac{Q}{A} = \frac{0{,}0065}{0{,}0074} = 0{,}88 \text{ m/s} \rightarrow V\ min \leq V \leq V\ ma$$

$$= 0{,}3 \text{ m/s} \leq 0{,}88 \text{ m/s} \leq 4{,}5 \text{ m/s (OK)}$$

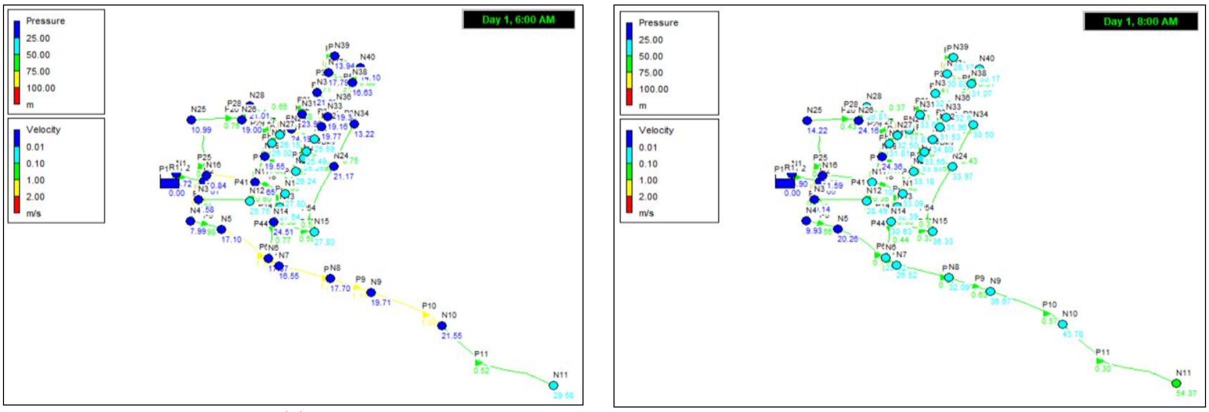
Pressure Loss:

Head Elev. (Upstream) $= h_1 + \frac{P_1}{\gamma_w} + \frac{{V_1}^2}{2g} = 608 + 0 + 0 = 608$ m

Head Elev. (Downstream) $= h_2 + \frac{{V_2}^2}{2g} + h_L - \frac{P_2}{\gamma_w} = 602 + \frac{0{,}88^2}{2 \times 9{,}81} + 0{,}323 - \frac{P_2}{\gamma_w} = 602.36$ m

Pressure of *Node 1* = Head Elev. (Upstream) – Head Elev. (Downstream)
= 608 m – 602.36 = 5.64 m (manual)
= 5.68 m = 0.06 MPa (software result simulation)

According to the standard[16], the minimum allowable pressure is 0.5 atm (0.05 MPa) – 1 atm (0.1 MPa) at the farthest service point, and the maximum pressure for HDPE PE100 SDR17 S8 PN10 pipes is 10 Bar (or 1 MPa, approximately 1000 kPa). The pressure calculation results at node 1 have met the design requirements. Simulations were conducted during peak hours (FP = 1.75) and average hours (FP = 1.00), as well as network performance during high demand (peak hours) and normal demand (off-peak hours) to ensure the system can handle water demand fluctuations. The simulation was performed on a pipe network consisting of 40 nodes, 1 reservoir, and 54 pipe segments. The visualization of the distribution pipe network based on the software simulation is shown in the following figure.



(a) (b)

Figure 5. SB.1 Distribution Pipeline Network; (a) Peak time; (b) Average time

Based on the analysis during peak hours, the flow velocity in all pipe elements has met the established standards. The highest velocity was observed in Pipe P7 at 1.67 m/s, while the lowest velocity was in Pipe P11 at 0.52 m/s. During average hours, the highest velocity in Pipe P7 was 0.96 m/s, and the lowest velocity occurred in Pipe P11 at 0.3 m/s. According to the Ministry of Public Works and Housing (Permen PUPR) [16], the standard velocity that must be achieved is between 0.3 m/s and 4.5 m/s.

During peak hours, the water pressure across all pipe elements has met the standard. The highest pressure occurs at Node 18 (Service Node) with a value of 26.59 m (0.27 MPa). Meanwhile, the lowest pressure occurs at Node 1 (Connection Node) with a value of 5.68 m (0.06 MPa) and at Node 34 (Service Node) with a value of 9.27 m (0.09 MPa). According to the Regulation of the Ministry of Public Works and Public Housing (Permen PUPR)[16], the minimum pressure that must be met ranges from 0.5 atm (0.05 MPa) to 1 atm (0.1 MPa) at the farthest service point, while the maximum pressure for HDPE PE100 SDR17 S8 PN10 pipes is 10 Bar (or 1 MPa, approximately 1000 kPa).

On average, the water pressure occurring across all pipe elements has also met the standards. The highest pressure occurs at Node 11 (Service Node) at 52.37 m (0.52 MPa), while the lowest pressure is at Node 1 (Connection Node) at 5.89 m (0.06 MPa) and Node 5 (Service Node) at 20.01 m (0.20 MPa). According to regulations, the minimum pressure that must be met ranges between 0.5 atm (0.05 MPa) and 1 atm (0.1 MPa) at the furthest service reach point.

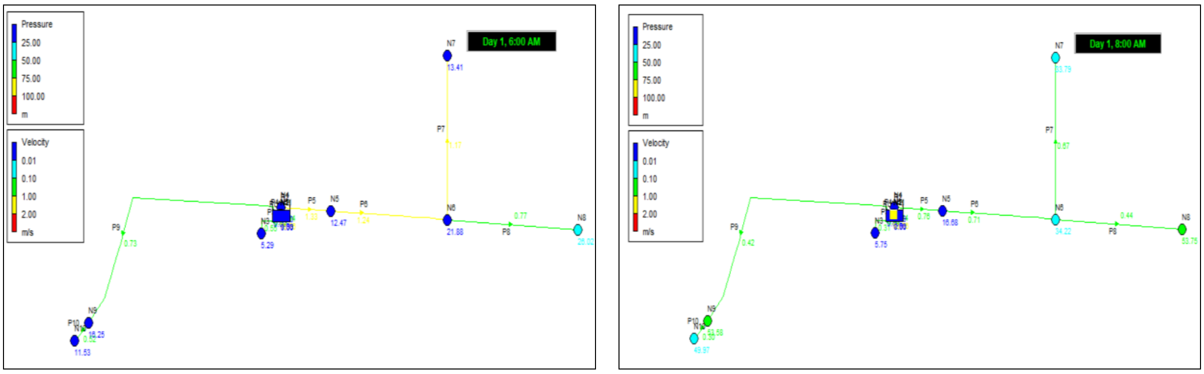
Table 4. SB.1 Distribution Pipe Requirement

No.	Pipe Diameter (inch)	SB.1		SB.2		Notes
		Total (m)	Rounded (m)	Total (m)	Rounded (m)	
1.	4”	113.30	114	3.00	3	
2.	2 ½”	143.80	144	122.00	123	
3.	2”	238.30	239	886.90	887	
4.	1 ½”	690.80	691	-	-	
5.	1 ¼”	1120.20	1121	-	-	
6.	1”	1098.80	1099	-	-	
7.	¾”	361.30	362	789.60	790	
8.	½”	1077.30	1078	451.80	452	

Souce: Analysis Result

At the SB.2 location, the simulation was conducted on a pipeline network consisting of 10 nodes, 1 reservoir, and 10 pipe segments. The image below shows the visualization of the distribution pipeline network using simulation software. The simulation results during peak hours (FP = 1.75) and average hours (FP = 1.00) were analyzed to examine changes in pressure and flow over time. The performance of the network during high demand (peak) and normal demand (non-peak) periods was evaluated to ensure the system can handle fluctuations in water demand.

The simulation results during peak hours show that the water velocity in all pipes meets the standard. The highest velocity was obtained in Pipe P5 at 1.33 m/s, while the lowest velocity was in Pipe P10 at 0.52 m/s. According to the standard, the required velocity should be between 0.3 m/s and 4.5 m/s. Additionally, the water pressure in the pipes also meets the standard. The highest pressure was at Node 11 (Pump output) at 76.76 m (0.77 MPa), and at Node 8 (Service node) at 26.02 m (0.26 MPa). The lowest pressure was at Node 2 (Connection node) at 1.98 m (0.02 MPa), and at Node 3 (Service node) at 5.29 m (0.05 MPa).



(a) (b)

Figure 6. SB.2 Distribution Pipeline Network; (a) Peak time; (b) Average time

The simulation results at the average hour show that the water velocity in the pipes has met the standards. The highest velocity is in Pipe P5 at 0.76 m/s, while the lowest velocity is in Pipe P10 at 0.3 m/s. The highest pressure is at Node 11 (Pump output) at 93.76 m (0.94 MPa), and at Node 8 (Service node) at 53.75 m (0.54 MPa). On the other hand, the lowest pressure is at Node 2 (Connection node) at 1.99 m (0.02 MPa) and at Node 3 (Service node) at 5.75 m (0.06 MPa). These results are in accordance with PUPR Regulation[16], which stipulates that the minimum pressure to be met is 0.5 atm (0.05 MPa) to 1 atm (0.1 MPa) at the farthest service point.

4. Reservoir and Pump

Reservoir and pump planning aims to ensure the availability of an adequate water supply in accordance with the planned needs. The selection of pumps, transmission pipes, and designed reservoirs must be able to support efficient water distribution. Pumps are planned to transport water from bore wells to the reservoir and/or from the reservoir to service zones when elevation is not feasible, or by considering the required flow capacity and head. Transmission pipes are designed to convey water from bore wells to the reservoir, minimizing pressure losses along the pipeline. Reservoirs are calculated to meet daily water demands and reserves, while also considering demand fluctuations. Each component is planned with attention to technical and operational efficiency, ensuring that the water supply system can operate optimally and sustainably.

4.1 Design of Pump

After selecting the pipe dimensions and running the flow simulation with the help of software, it was found that there is negative pressure at Node 9 and Node 10, with a pressure head value of -63.23 m. Under these conditions, a pump is needed to increase the pressure to ensure the system meets service requirements. The solution used is to select a pump with a head capacity greater than the negative pressure head value. When choosing a pump, several factors must be considered, including the head capacity and the discharge capacity that the pump can deliver. This is to avoid inaccuracies in solving the problem.

A pump with a head capacity of 75 m was added for testing.
P-pressure lost = Head Pumpa + P(-) = 75 m + (-63.23) = 11.77 m (Ok)
The pressure head curve against the resulting flow velocity represents the required pressure head to resolve the water distribution issue in the study area.

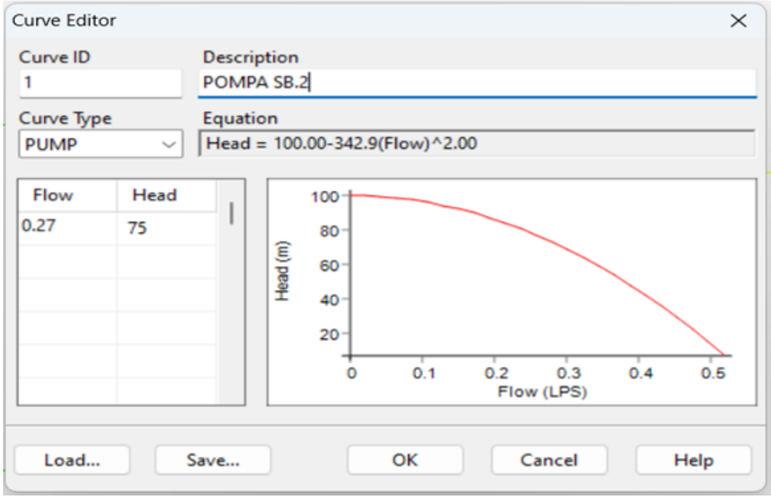


Figure 7. Pump Curve [Head and velocity]

4.2 SB.1 Reservoir Capacity

Reservoir SB.1 is located at an elevation of +606 meters above sea level. The reservoir capacity planning is based on peak-hour water demand of 6.5 l/s, an average demand of 3.71 l/s, and daily water usage fluctuations over 24 hours to ensure a sufficient water supply throughout the day. According to previous studies on water availability, the potential clean water supply from SB.1 is 6.99 l/s, which is sufficient to meet both peak-hour and average demand in the service area. With this capacity, the system can accommodate fluctuations in water usage and ensure a sustainable, uninterrupted supply.

Table 5. SB.1-Reservoir Capacity Analysis

Pump scheme	Time (hours)	Inlet		Outlet				Reservoir Capacity
		Q (m3/h)	Cum. Volume (m3)	FP	Q average (m3/h)	Q (m3/h)	Cum. Volume (m3)	Volume (inlet –outlet)
On	1.00	25.17	25.17	0.64	13.36	8.50	8.50	16.67
On	2.00	25.17	50.34	0.64	13.36	8.50	17.00	33.35
On	3.00	25.17	75.51	0.64	13.36	8.50	25.49	50.02
On	4.00	25.17	100.68	0.64	13.36	8.50	33.99	66.69
On	5.00	25.17	125.86	1.50	13.36	20.04	54.03	71.82
On	6.00	25.17	151.03	1.50	13.36	20.04	74.08	76.95
On	7.00	25.17	176.20	1.75	13.36	23.38	97.46	78.74
Off	8.00	0.00	176.20	1.75	13.36	23.38	120.84	55.36
Off	9.00	0.00	176.20	1.00	13.36	13.36	134.20	42.00
Off	10.00	0.00	176.20	1.00	13.36	13.36	147.56	28.63
Off	11.00	0.00	176.20	1.00	13.36	13.36	160.93	15.27
Off	12.00	0.00	176.20	1.00	13.36	13.36	174.29	1.91
On	13.00	25.17	201.37	0.64	13.36	8.50	182.79	18.58
On	14.00	25.17	226.54	0.64	13.36	8.50	191.28	35.26
On	15.00	25.17	251.71	0.64	13.36	8.50	199.78	51.93
On	16.00	25.17	276.88	1.00	13.36	13.36	213.14	63.74
On	17.00	25.17	302.05	1.75	13.36	23.8	236.53	65.53
On	18.00	25.17	327.23	1.75	13.36	23.38	259.91	67.32
On	19.00	25.17	352.40	1.00	13.36	13.36	27327	**79.13**
Off	20.00	0.00	352.40	1.00	13.36	13.36	286.63	65.77
Off	21.00	0.00	352.40	0.64	13.36	8.50	295.13	57.27
Off	22.00	0.00	352.40	0.64	13.36	8.50	303.63	48.77
Off	23.00	0.00	352.40	0.64	13.36	8.50	312.13	40.27
Off	00.00	0.00	352.40	0.64	13.36	8.50	320.62	31.77

Source: Analysis Result

Based on the analysis of the SB.1 reservoir capacity, the reservoir dimensions can be calculated as follows:

Vol. of Reservoir SB.1 = Cumulative inlet volume - Cumulative outlet volume
= 352,4 – 273,27 = 79,13 m3

Vol. of Reservoir SB.1 = (79,13 x 20%) + 79,13
= 94,96 $m^3 \approx 95\ m^3$

Thus, the reservoir dimensions are:

Vol. Capacity = Long [P] x Width [L] x Height [T] = PLT, if T = 3 meter
95 m^3 = P x L x 3
95 m^3 ≤ [8.00 m x 4.00 m x 3.00 m]

4.3 SB.2-Reservoir Capacity

Reservoir SB.2 is planned to be located at an elevation of +685 meters above sea level. The reservoir capacity planning is based on a peak-hour water demand of 5.37 l/s, an average demand of 3.07 l/s, and daily water usage fluctuations over 24 hours to ensure a sufficient water supply throughout the day. According to previous studies, the available water potential is 5.69 l/s, which is sufficient to meet both the peak-hour and average demand in the service area

Table 6. SB.2-Reservoir Capacity Analysis

Pump scheme	Time (hours)	Inlet		Outlet				Reservoir Capacity
		Q (m3/h)	Cum. Volume (m3)	FP	Q average (m3/h)	Q (m3/h)	Cum. Volume (m3)	Volume (inlet –outlet)
On	1.00	20.48	20.48	0.64	11.04	7.02	7.02	13.46
On	2.00	20.48	40.96	0.64	11.04	7.02	14.05	26.91
On	3.00	20.48	61.44	0.64	11.04	7.02	21.07	40.37
On	4.00	20.48	81.92	0.64	11.04	7.02	28.10	53.82
On	5.00	20.48	102.40	1.50	11.04	16.57	44.67	57.73
On	6.00	20.48	122.88	1.50	11.04	16.57	61.23	**61.65**
On	7.00	25.17	143.36	1.75	11.04	19.33	80.56	43.47
Off	8.00	0.00	143.36	1.75	11.04	19.33	99.89	36.62
Off	9.00	0.00	143.36	1.00	11.04	11.04	110.93	32.42
Off	10.00	0.00	143.36	1.00	11.04	11.04	121.98	21.38
Off	11.00	0.00	143.36	1.00	11.04	11.04	133.02	10.33
Off	12.00	0.00	143.36	1.00	11.04	11.04	144.07	-0.71
On	13.00	20.48	163.84	0.64	11.04	7.02	151.09	12.74
On	14.00	20.48	184.32	0.64	11.04	7.02	158.12	26.20
On	15.00	20.48	204.80	0.64	11.04	7.02	165.14	39.65
On	16.00	20.48	225.28	1.00	11.04	11.04	176.19	49.09
On	17.00	20.48	245.76	1.75	11.04	19.33	195.52	50.24
On	18.00	20.48	266.24	1.75	11.04	19.33	214.84	51.39
On	19.00	20.48	286.72	1.00	11.04	11.04	225.89	60.83
Off	20.00	0.00	286.72	1.00	11.04	11.04	236.93	49.78
Off	21.00	0.00	286.72	0.64	11.04	7.02	243.96	42.76
Off	22.00	0.00	286.72	0.64	11.04	7.02	250.98	35.73
Off	23.00	0.00	286.72	0.64	11.04	7.02	258.01	28.71
Off	00.00	0.00	286.72	0.64	11.04	7.02	265.03	21.68

Source: Analysis Result

Based on the analysis of the SB.1 reservoir capacity, the reservoir dimensions can be calculated as follows:

Vol. of Reservoir SB.2 = Cumulative inlet volume - Cumulative outlet volume
= 122.88 – 61.23 = 61.65 m3

Vol. of Reservoir SB.2 = (61.65 x 20%) + 61.65
= 73.98 $m^3 \approx 75\ m^3$

Thus, the reservoir dimensions are:

Vol. Capacity = Long [P] x Width [L] x Height [T] = PLT, if T = 3 meter
80 m^3 = P x L x 3
80 m^3 ≤ [7.50 m x 3.50 m x 3.00 m]

The integration between capacity planning and pump operations with reservoir capacity is a crucial aspect in the management of an effective and sustainable clean water supply system. Good integration and optimal acceleration can lead to efficiency in the utilization of clean water and the energy required to operate the pumps. Providing intervals and rest periods for the pumps is a wise measure to extend their service life and maintain the health of groundwater potential, as it prevents continuous exploitation.

Good planning and arrangement of the network provide sufficient energy efficiency in the hydraulic process of water flow from the reservoir to the service zones. Optimal network arrangement and simulation allow for the use of pipes with appropriate and non-excessive dimensions, ensuring that the hydraulic energy utilized is both effective and optimal. The reservoir optimization pattern, as presented in the table, serves as an excellent option because the planned dimensions accurately refer to the required capacity, avoiding excess and achieving optimality.

IV. CONCLUSION

Based on the study and analysis conducted, it can be concluded that the geological and geographical conditions of the study area present several fundamental challenges related to the network model and operational pattern to be simulated. The elevation difference between the potential resources to be utilized and the service zone requires selecting the most optimal and effective pattern, ensuring that the dimensions of the supporting infrastructure are simulated optimally.

The population projection based on 10 years of data in the study area, using the selected arithmetic method, estimates a population of 9,574 people in 2032. Based on the water demand calculations for the study area in 2032, the total average water demand is 7.82 l/s, the maximum daily demand is 9.38 l/s, and the peak hour demand is 13.28 l/s. The available groundwater potential at SB.1 and SB.2, according to previous studies, is still sufficient to meet the average and peak hour demand for each service zone. This indicates that the available water resources can fulfill demand until 2032.

The analysis of distribution pipe network requirements, based on simulation results, indicates that SB.1 requires pipes with diameters of 4", 2 ½", 2", 1 ½", 1 ¼", 1", ¾", and ½". Meanwhile, SB.2 requires pipes with diameters of 4", 2 ½", 2", ¾", and ½". The recommended pipe type is HDPE (High-Density Polyethylene) SDR17 PN10, as it is flexible and resistant to pressure, making it suitable for the study area's extreme elevation differences. The optimization results for the planned reservoir capacity at the two proposed locations include 95 m^3 for SB.1 and 80 m^3 for SB.2.

Good planning and arrangement of the network provide sufficient energy efficiency in the hydraulic process of water flow from the reservoir to the service zones. Optimal network arrangement and simulation allow for the use of pipes with appropriate and non-excessive dimensions, ensuring that the hydraulic energy utilized is both effective and optimal. The reservoir optimization pattern, as presented in the table, serves as an excellent option because the planned dimensions accurately refer to the required capacity, avoiding excess and achieving optimality.

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