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DOMESTIC WASTEWATER PIPING NETWORK PLANNING AND TECHNOLOGY RECOMMENDATIONS FOR WASTEWATER TREATMENT CASE STUDY: THE AMBARITA AREA, SAMOSIR REGENCY, NORTH SUMATRA

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Abstract

Lake Toba is one of ten programs the Ministry of Tourism in the Republic Indonesia, as a priority tourism area because of its unique potential. The Ambarita is one of 16 villages that are priority areas in Simanindo Subdistrict, Samosir Regency. At present, the quality of Lake Toba's waters has been polluted, the pollution is caused by domestic waste water which is discharged directly without prior treatment. The purpose of this study is to plan a domestic wastewater pipeline network and recommendations for wastewater treatment plants with a centralized system in the Ambarita. This study uses a quantitative data analysis method by using a population projection approach with maximum capacity to be planned in the draft MasterPlan of Ambarita and literature study for WWTP technology recommendations. The results showed that the total discharge of wastewater in the Ambarita was 0,06495 m3/second and the results of laboratory tests showed the parameters that exceeded quality standard were free chlorine and fecal coli. Piping network planning in the Ambarita uses concrete type pipes with diameters of 100 mm, 125 mm, 150 mm, 200 mm and 250 mm. The selected Waste Water Treatment Technology recommended is anaerobic-aerobic biofilter technology with removal efficiency for fecal coli and free chlorine are 99.9% and 65%.

Keywords: *concrete type pipes; anaerobic-aerobic biofilter; free chlorine; fecal coli.*

INTRODUCTION

Lake Toba is a volcanic lake located in North Sumatra. The potential for the uniqueness of Lake Toba is that there is an island in the middle of the lake called Samosir Island so that its uniqueness makes Lake Toba one of the ten programs of the Ministry of Tourism Republic Indonesia, as a priority tourist area. According to the President of the Republic of Indonesia, Lake Toba has 28 tourist points that can be developed into world-class tourist attractions. This development was carried out in 2019 and is planned to be completed in 2020 under the order of the President Republic Indonesia (Judisseno, 2019).

To support National Tourism Strategic Area (KSPN) program, the development of the Samosir especially in The Ambarita Area has many considerations in its implementation, especially in environmental issues such as the quantity and quality of Lake Toba waters in supporting tourism activities and the surrounding community. The clean water that contaminated biologically or chemically can cause the negative impact on the public health (Sari, Madonna, et al., 2018). However, currently the water quality of Lake Toba has been polluted, which causes the waters of Lake Toba to smell and become turbid. Turbidity caused by high biological activity such as algae, phytoplankton, zooplankton which is triggered by an increase in phosphorus levels (Sari, Sugiana, et al., 2018). According to the Ministry of Marine Affairs and Fisheries in 2018, one of the contributing sources of pollutant sources for Lake Toba waters comes from floating net cage activities which cause high phosphorus content due to the use of excess fish feed, which is 941.1 tons / year from floating net cage fish cultivation and phosphorus pollutant in the river that enters Lake Toba amounting to 25,334 tons / year (Kartamihardja et al., 2015).

Other sources of pollutant waters of Lake Toba are caused by discharges of domestic wastewater from household activities and other public facilities. The occurrence of siltation, the quality of river water becomes polluted and the flow of the river water is stopped caused by the accumulation of the waste is a condition where the biophysical quality of the river decreases, which can be caused by the waste that is disposed of by the community into the river (Sari, Sugiana, et al., 2018). Besides, the water body is passed by various kinds of waste like domestic and industrial waste, causing water bodies to contain nutrients, which are phytoplankton food (Sari, Sugiana, et al., 2018). Disposal of domestic wastewater in the Lake Toba area, especially Ambarita, still uses a local treatment system where there is no transportation to serve it, so that domestic wastewater is discharged directly into septic tanks, lakes, rivers, or irrigation canals without prior treatment which causes pollution of Lake waters. Toba. According to the 65.3% of the people who have a septic tank, 33.6% have not met technical standards and only 23.8% have drains other than feces. The Ambarita area will become a City Service Center covering lodging, tourism, as well as supporting facilities and infrastructure. However, seeing the sanitation conditions in the tourist area of Lake Toba, it is necessary to build a centralized system of Domestic Wastewater Treatment Plants.

Wastewater Treatment Plants will be built around the lake which has a tourism and housing area with an offsite system (centralized). The various topographical conditions of the Ambarita Area, the Domestic Wastewater Treatment Plant will be located in an area that has the lowest contour and utilizes a gravity system. The wastewater drainage system with gravity must be supported by a pipeline network for channel domestic wastewater to a centralized WWTP which will be planned in the Ambarita area.

Pollution of water bodies, decreasing level of public health encourages the establishment of an integrated wastewater treatment system. The system offered is a domestic wastewater management system which includes the distribution and treatment of domestic wastewater, in the form of gray water and black water (Pratiwi & Purwanti, 2015). The aim of this research is to design a domestic wastewater pipeline in the Ambarita area and make recommendations for the most appropriate WWTP technology to be able to treat domestic wastewater in the area so as not to pollute Lake Toba. This research becomes a new input in relation to port development in Ambarita, this research not only makes the design of the waste water pipe network but also provides technology recommendations that can be used in WWTP in the area.

RESEARCH METHODS

The pipeline network planning and recommendations for domestic WWTP are carried out based on the results of several field applications that are in accordance with existing conditions. This research was conducted in the Ambarita area, Simanindo District, Samosir Regency, North Sumatra from December 2019 to February 2020. Sources of data used in this study come from primary data, namely the amount of wastewater, sources of wastewater, conditions of MCK, WWTP, soil elevation and documentation to support the research, while secondary data includes topography, lake water quality parameters Toba, population, average water consumption, location map. The method used in this research is using a descriptive quantitative data analysis method in designing and recommending domestic WWTP (Nasoetion et al., 2017). The stages of the research method as a whole can be seen in Figure 1.

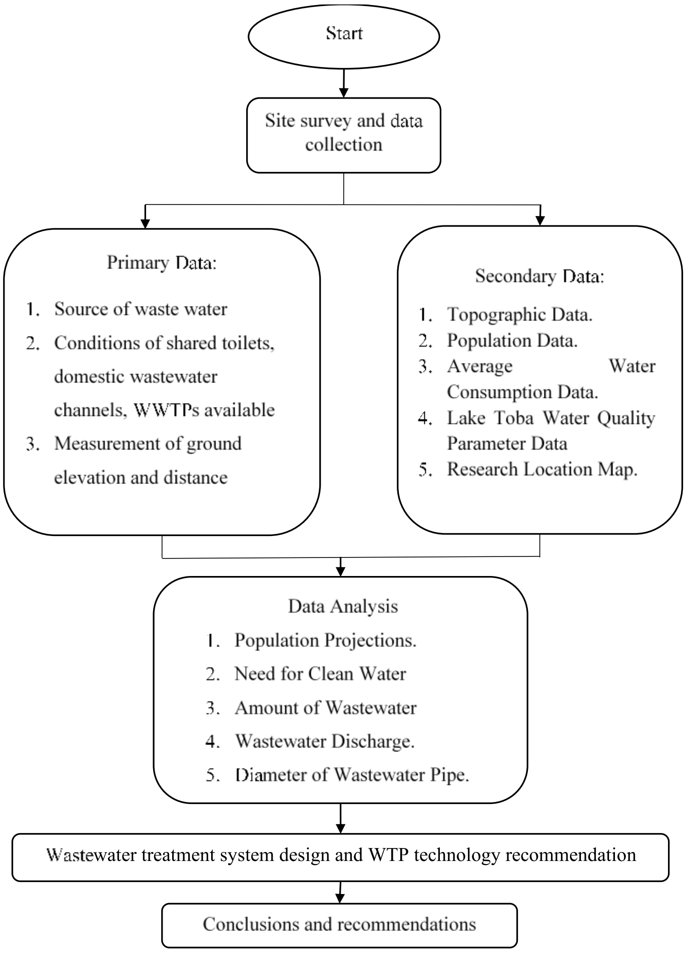

Figure 1.
Research Flowchart

The calculated and analyzed data is processed to obtain a domestic wastewater pipeline network design and a recommendation for WWTP technology using the following methods:

A. Population Projection

Indonesia is the fourth country with biggest population in the world (Sari, Madonna, et al., 2018). The calculation of population projections will affect domestic clean water needs. The population projections of the Simanindo Priority Area are divided into 2 types, namely natural projections and progressive projections. Natural projection is a natural population growth projection which has not been influenced by external factors that will affect population development in the area. Meanwhile, a progressive projection is a population projection that looks at population development if population migration is expected to support. Unfortunately, high population density can lead to natural disaster and disturbed ecosystem balance (Sari, Sugiana, et al., 2018).

There are several methods that can be used to analyze the development of natural population, namely:

1. Least Square Methode

   This method is carried out if the amount of data is odd by using the following equation:

$$y_{(t)} = a + b\,x$$

$$a = \frac{\sum y_i}{n}$$

$$b = \frac{\sum y_i u_i}{\sum u_i^2}$$

   Annotation:
   yi = Total population in year-i
   ui = Multiplier variable
   x = The period of the year between the projection year and the current year
   ui = 0

2. Arithmetic method

   This method is used if periodic data shows the number of additions that are relatively the same each year. This usually occurs in cities with small areas, low economic growth rates and not too fast urban development. The formula for this method is:

$$y_{(t)} = y_n + g.x$$

   Annotation:
   yn = Total population in final data
   x = The period between the projection year and the final data year
   g = Average annual population growth

$$= \frac{y_{\text{akhir data}} - y_{\text{awal data}}}{\text{jumlah data}}$$

3. Geometric Method

   This method is used if the population data shows a rapid increase from year to year. The formula for this method is:

$$y_{(t)} = y_n\,(1 + r)^x$$

$$r = \left(\frac{y_n}{y_o}\right)^{\frac{1}{n}} - 1$$

Annotation:
y(t) = Projected population in a certain year
yn = Total population in final data
x = The period between the projection year and the final data year
r = Population growth rate
n = The amount of data

4. Linear Regression Method

$$y = a + bx$$

$$a = \frac{\Sigma y \Sigma x^2 - \Sigma x \Sigma (xy)}{N \Sigma x^2 - (\Sigma x)^2}$$

$$b = \frac{N \Sigma (xy) - \Sigma x \Sigma y}{N \Sigma x^2 - (\Sigma x)^2}$$

5. Exponential Method

$$y = a\, e^{b x_n}$$

$$\ln a = \left(\frac{1}{N}\right)(\Sigma \ln y - b\, \Sigma x)$$

$$b = \frac{N\, \Sigma (x \ln y) - (\Sigma x\, \Sigma \ln y)}{N\left(\Sigma x^2\right) - (\Sigma x)^2}$$

6. Logarithmic Method

$$y = a + b \ln x$$

$$a = \frac{1}{N}\left[\Sigma y - b\, \Sigma(\ln x)\right]$$

$$b = \frac{N\, \Sigma (y \ln x) - \Sigma y \Sigma \ln x}{N\, \Sigma (\ln x)^2 - (\Sigma \ln x)^2}$$

To determine the most appropriate method to be used in planning, it is necessary to calculate the correction factor, standard deviation, and the state of future developments in the city. The correlation, r, can be calculated using the formula:

$$R^2 = 1 - \frac{SSE}{SST}$$

$$R^2 = 1 - \frac{\Sigma (P_n - P)^2}{\Sigma (P_n - P_r)^2}$$

Annotation:
R^2 = Correlation factor
P_n = Total population at year n
P_r = average population of known data
P = Projected population based on calculations the regression method done

The correlation criteria are as follows:
1. r < 0, strong correlation, but negative value and relationship between the two variables is inversely proportional.
2. r = 0, both data have no relationship.

3. r > 1, strong correlation, positive value and the relationship between the two variables is directly proportional.

The standard deviation can be calculated using a formula:

$$STD = \left[\frac{\Sigma (P_n - P)^2 - \left[\Sigma \frac{(P_n - P)^2}{n} \right]}{n} \right]^{\frac{1}{2}}$$

Annotation:
STD = Standard deviation of known data
n = Amount of known data

The projection method chosen is the method with the lowest standard deviation value and the largest correlation coefficient.

B. Need for Clean Water

a) Domestic Water Needs

Domestic water needs are met in two ways, namely House Connections and Public Taps. For now, the Ambarita area still serves the planning area at 0%. It is planned to increase to 91%. Based on data (Badan Pusat Statistik, 2018), the existing population of Ambarita in 2018 is 1043 people and is included in the rural category. The ratio between the number of residents served by Domestic Connection and Public Taps for rural areas is 70:30. Standard drinking water needs of:

- Home connection: 100L / person /day
- General faucet: 30 L / person / day

The calculation of domestic water demand can be seen in the equation below (Fatimah et al., 2014).

Domestic Water Need = Total Population x Water Needs per Capita

Per capita water needs will differ in each city depending on the social and economic level of the community. The minimum basic requirement for water consumption per person is 121 liters per day. These needs are needed in the needs of drinking, cooking, washing clothes, bathing, cleaning the house and the needs of worship. According to Poedjastanto, Indonesia's minimum basic need is 70 liters / person / day (Pusat Komunikasi Publik Kementrian PUPR, 2007).

b) Non-Domestic Water Needs

Non-domestic water needs are water needs that include urban (social and public) facilities in the planning area. Urban facilities located in Ambarita include education, health, worship and recreation. The calculation of non-domestic water demand is as follows (Fatimah et al., 2014).

Total Non-Domestic Water Needs = ∑Water Requirement for Industry + Water Needs for Trade and Services + Water Needs for Other Economic Activities.

The water requirement of each activity is the multiplication of the number of units and the water requirement per unit of activity.

C. Domestic Wastewater Discharge

In calculating the volume of domestic wastewater, it is necessary to know the volume of clean water requirements. The need for clean water is expressed as the peak hour

discharge which is the amount of water during the greatest use in 24 hours (Martono, 2015).
The peak hour discharge is calculated by the formula below.

$$Q_{jp} = f_{jp} \text{ x } Qr$$

Annotation:
Qjp = peak hour discharge (m^3/s)
Qr = average discharge (m^3/s)
fjp = peak hour factor

The peak hour discharge states the amount of clean water needed. Wastewater discharge states the amount of wastewater or waste produced by the community during a certain period of time. According to (Martono, 2015) wastewater discharge can be calculated in the following formulation below.

$$\boldsymbol{Q_{ab} = 80\% \times Q_{jp}}$$

Annotation:
Q_{ab} = wastewater discharge (m^3/s)
The minimum wastewater discharge (Qmin) is the discharge of wastewater when using minimum water. The minimum discharge calculation requires equivalent population data (PE) and average wastewater discharge ($Q_{r\ ab}$). Equivalent population (PE) is the amount of organic waste decomposed from household and commercial activities (Martono, 2015).

$$Qmin = 0.2 \times \sum PE^{1.2} \times Q_{r\ ab}$$

$$Q_{r\ ab} = \frac{\sum \mathrm{Qab}}{\mathrm{n}}$$

$$PE = \frac{\mathrm{Qab}}{Qr}$$

Annotation:
Qmin = minimum wastewater discharge (m^3/s)
PE = equivalent population (people)
$Q_{r\ ab}$ = average wastewater discharge (m^3/s)
n = number of nodes in a wastewater distribution system

The peak discharge calculation can be obtained from the equation below.

$$Qpeak = Q_{r\ ab} \text{ x } fpeak$$

Annotation:
Qpeak = Peak discharge of wastewater (m^3/s)
$Q_{r\ ab}$ = Average water discharge waste (m^3/s)
F*peak* = Peak factor

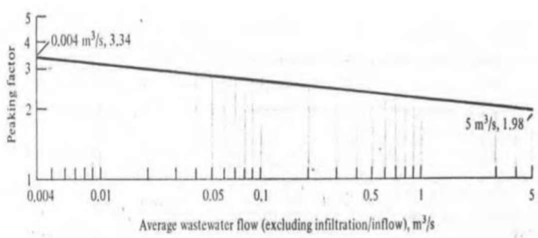

Figure 2.
Wastewater Peak Factor
Source : (Pratiwi & Purwanti, 2015)

The maximum wastewater discharge (Qmax) is the discharge of wastewater at the time of maximum water use. The Qmax calculation uses the maximum daily factor value data which is the ratio between the average water (Martono, 2015).

$$Qmaks = 5 \times PE^{0.8} \times fhm \times Qr$$

Annotation: fhm = max. daily factor (1,2)

D. Wastewater Channelling System

The design of the wastewater distribution system must first know the value of the initial full flow rate (initial Qfull). The initial full flow rate (initial Qfull) is calculated as follows (Martono, 2015).

$$Q_{full\ awal} = Qpeak \times \frac{\mathrm{Qfull}}{\mathrm{Qpeak}}$$

Determine the value of d/D. The d/D value is used to get the Qfull/Qpeak value. The Qfull/Qpeak values are obtained from the graph design of main sewers on **Figure 3**

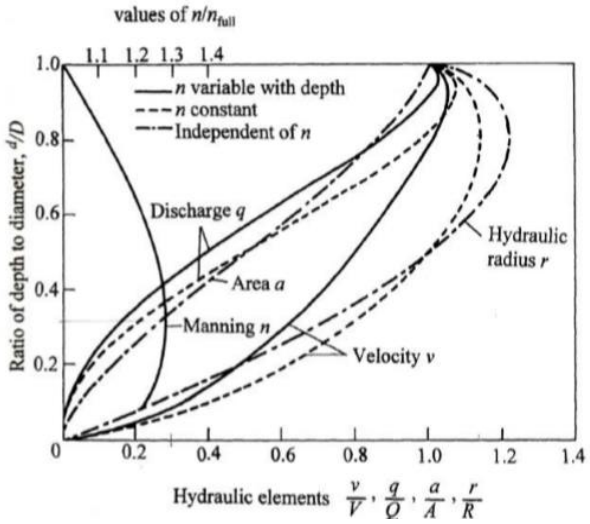

Figure 3.
Design of Main Sewers Graph
Source : (Martono, 2015)

The calculation of the dimensions of the sewerage is carried out after the initial full flow rate (initial Qfull) and full flow velocity (vfull) are assumed. The channel diameter is generated by the following formula (Martono, 2015).

$$D_{count} = \sqrt{\frac{4\ x\ (\frac{Qfull\ initial}{Vfull})}{\pi}}$$

Once the diameter is obtained, the hydraulic radius (R) can be calculated. The hydraulic radius is calculated using the following equation (Martono, 2015).

$$R = 0.25 \times D$$

Annotation:
D_{count} = diameter of the output channel calculation (mm)
$Q_{full\ Initial}$ = initial full flow rate (m^3/s)
V_{full} = full flow rate (assumed) (m/s)

R = hydraulic radius (mm)
D = channel diameter (mm)

The slope of the pipe (slope) is one of the factors that greatly affects the value of V_{full}. A minimum slope of the pipe is required so that the minimum flow speed is obtained by self-cleansing so that deposits do not occur in the sewerage. The pipe slope value can be an assumption provided that the value of V_{full} is not less than 0.6 m/s and not more than 3 m/s (Martono, 2015). Calculate the slope of the field using the following equation (Pratiwi & Purwanti, 2015).

$$S = \frac{\Delta H}{L}$$

Annotation:
S = Slope
Δ H = Different in height (m)
L = Long (m)

Full flow velocity (v_{full}) is the velocity of wastewater flow when the pipe is full. The full flow rate is calculated by the Manning equation (Martono, 2015).

$$\text{Vfull} = \frac{1}{n}\ x\ R^{\frac{2}{3}}\ x\ S^{\frac{1}{2}}$$

Annotation:
S = slope of pipe or channel (%)
n = Manning's coefficient of roughness
Vpeak = peak flow rate (m/s)
R = hydraulic radius

The full discharge of wastewater (Qfull) is the discharge of wastewater when the pipe is full. In addition, it is necessary to calculate the minimum water level (dmin) and minimum flow velocity (vmin). This was done so that the need for flushing could be identified in a waste water distribution system.

$$Qfull = \frac{1}{4}\ x\ \pi\ x\ D^2 \text{ x vfull}$$

$$d_{min} = \frac{dmin}{D}\ x\ D$$

Annotation:
Qfull = full discharge of wastewater (m^3/s)
dmin = minimum water level (mm)
D = channel diameter (mm)

The vmin/vfull value can be found using the dmin/D value in the design of main sewers graph **Figure 3.2.** The minimum flow velocity is given by the equation below.

$$\mathbf{V_{min}} = \frac{Vmin}{Vfull}\ x\ Vfull$$

RESULTS AND STUDY

A. Study Area Conditions

The Ambarita area is in position 2°40’92” east longitude and 98°49’47” South latitude. Slope conditions in the Ambarita area, which is included in the Simanindo Priority Area, have a slope of more than 40%, which is 24% of the total land area, which is difficult to build, while the easy-to-build areas that are on a slope of 0-8% are only around 18%. According to (Badan Pusat Statistik, 2018) The total population of the Ambarita area is 1043. The Ambarita area was selected as the study area because the Ambarita area is a priority tourist area whose population will increase rapidly due to tourism activities. This will affect the generation of domestic waste water, which, if not handled, will cause problems with Lake Toba water pollution. The problem of wastewater in the Ambarita area has yet to be addressed because there is no sewage system and WWTP.

B. Population Projections of the Ambarita Area

In the planning of this domestic wastewater distribution system, a planning area is determined which will be calculated based on the maximum capacity of the design that will be made in the Master Plan draft, namely the area that is the busiest center of tourism activities to residential housing. The population to be served is the population of progressive projections and the design period used in this planning is 20 years from 2020 to 2040 according to the planning in the Master Plan draft in **Figure 4**

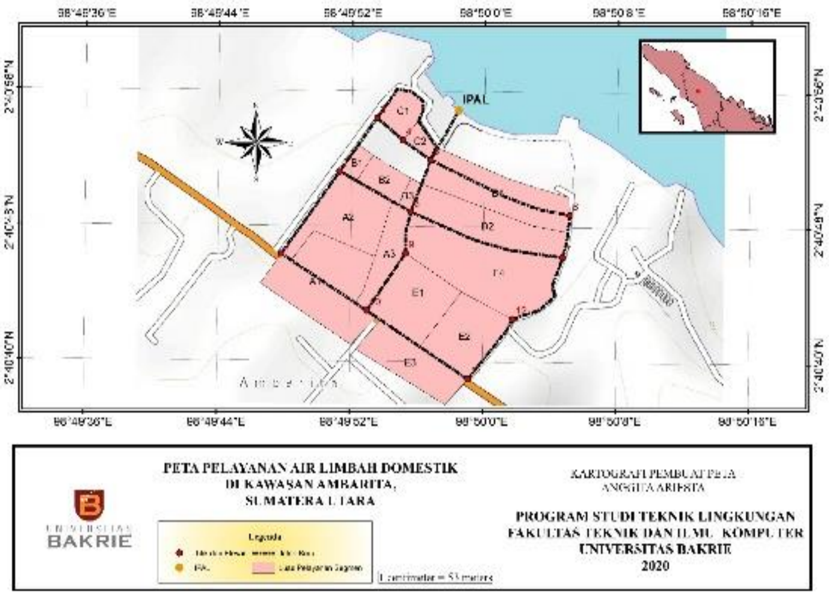

Figure 4.
***MasterPlan* Kawasan Ambarita**

From **Figure 4**, for the areas served, the number of parcels from each segment is obtained multiplied by the number of families (family cards) where each parcel consists of 5 family members, which can be seen in **Table 1**.

Table 1. Ambarita Area Population Per Segment

Segment	Point		Block	Cumulative Block	L	L eqivalent	$L_{cumulative}$	H_1	H_2	Block Area	Population		
											Population	Qumulative	Kumulatif
	From	to			(m)	(m)	(m)	(m)	(m)	(ha)	(population)	(population)	(/1000 population)
A	10	1	A1	A1	193,739	213	213	922	918	2,57	135	135	0,135
	1	2	A2	A1, A2	189,390	208	421	918	913	1,68	80	215	0,215
	10	8	A3	A1, A2, A3	207,930	229	650	922	911	0,80	95	310	0,310
B	2	3	B1	B1	121,800	134	784	913	910	0,21	25	335	0,335
	2	8	B2	B1, B2	154,467	170	954	913	911	0,48	20	355	0,355
	8	5	B3	B1, B2, B3	103,243	114	1068	911	907	0,14	25	380	0,380
C	3	4	C1	C1	64,233	71	1138	910	909	0,50	15	395	0,395
	4	5	C2	C1, C2	64,205	71	1209	909	907	0,22	30	425	0,425
D	6	5	D1	D1	285,251	314	1523	908	907	1,95	115	540	0,540
	8	7	D2	D1, D2	303,285	334	1856	911	907	1,69	20	560	0,560
E	10	9	E1	E1	129,765	143	1999	922	914	1,61	70	630	0,630
	11	12	E2	E1,E2	140,679	155	2154	924	916	1,49	140	770	0,770
	11	10	E3	E1,E2,E3	230,908	254	2408	924	922	1,34	130	900	0,900
	12	7	E4	E1, E2, E3, E4	168,800	186	2593	916	910	2,98	145	1045	1,045
	7	6			79,531	87	2681	910	908	0,00	0	1045	1,045
WWTP	5	WWTP			106,647	117	2711	907	904	2,98	1045	2090	2,090

Annotation:

1. Segment = Division of the area to be served is divided into several segments
2. Block = A segment divided into service blocks
3. L = pipe length (m)
4. L equivalent = length of straight pipe (m)
5. Cumulative L = Addition of the equivalent length before and after (m)
6. H1 = initial height (m)
7. H2 = the final height (m)
8. Block area = Area per block (ha)

C. Clean Water Needs and Wastewater Projection

Wastewater discharge is the need for clean water from the total domestic and non-domestic water needs that people use every day. Of this water requirement, about 80% will become domestic wastewater. The Ambarita area is included in the rural category so that the need for house connections is 95 l/person/day and non-domestic water needs such as health, education, recreation and sports facilities seen from the planning needs to be served in the Ambarita Area Master Plan draft and non-standard water needs. -Domestic PU Cipta Karya. Calculation of water needs for berish and wastewater can be seen in **Table 2 and Table 3.**

Table 2. Calculation of Clean Water and Wastewater Needs (1)

Qam							Qam qumulative	ΔHt	St	Sp	Fp	Qr of waste water
Domestic		Non-Domestic				Total						
L/s	(m^3/s)	Type	non domestic needs	L/s	(m^3/s)	(m^3/s)	(m^3/s)	m	m/m	m/m		(m^3/s)
0,148	0,00015	District Scale Education	15	0,02	0,00002	0,00017	0,00017	4	0,019	0,019	2,1	0,00014
0,088	0,00009	District scale health and District Scale Education	1215	1,13	0,00113	0,00121	0,00138	5	0,024	0,024	2,1	0,00111
0,104	0,00010	Office	1200	1,32	0,00132	0,00142	0,00281	11	0,048	0,048	2,1	0,00225
0,027	0,00003	Office	1200	0,35	0,00035	0,00037	0,00318	3	0,022	0,022	2,1	0,00255
0,022	0,00002	-	-	0,00	0,00000	0,00002	0,00321	2	0,012	0,012	2,1	0,00256
0,027	0,00003	-	-	0,00	0,00000	0,00003	0,00323	4	0,035	0,035	2,1	0,00259
0,016	0,00002	District Scale Education	15	0,00	0,00000	0,00002	0,00325	1	0,014	0,014	2,1	0,00260
0,033	0,00003	-	-	0,00	0,00000	0,00003	0,00329	2	0,028	0,028	2,1	0,00263
0,126	0,00013	District Scale Education	15	0,02	0,00002	0,00015	0,00343	1	0,003	0,003	2,0	0,00275
0,022	0,00002			0,00	0,00000	0,00002	0,00345	4	0,012	0,012	2,0	0,00276
0,077	0,00008	Sports Stadium	1200	0,97	0,00097	0,00105	0,00450	8	0,056	0,056	2,0	0,00360

Table 3.
Calculation of Clean Water and Wastewater Needs (2)

Qam							Qam qumulative	ΔHt	St	Sp	Fp	Qr of waste water
Domestic		Non-Domestic				Total						
L/s	(m^3/s)	Type	non domestic needs	L/s	(m^3/s)	(m^3/s)	(m^3/s)	m	m/m	m/m		(m^3/s)
0,154	0,00015	District Scale Education and Office	1215	1,97	0,00197	0,00212	0,00663	8	0,052	0,052	2,0	0,00530
0,143	0,00014	District Scale Education	15	0,02	0,00002	0,00017	0,00679	2	0,008	0,008	2,0	0,00543
0,159	0,00016	Office	1200	2,01	0,00201	0,00217	0,00896	6	0,032	0,032	1,9	0,00717
0,000	0,00000	-	-	0,00	0,00000	0,00000	0,00896	2	0,023	0,023	1,9	0,00717
1,149	0,00115	-	-	7,82	0,00782	0,00896	0,01793	3	0,026	0,026	1,8	0,01434

Annotation:

1. Qam = Discharge of clean water requirements (m^3/sec)
2. ΔHt = Difference between initial and final heights (m)
3. St = Sp = slope (m)
4. Fp = Peak factor
5. Qr of waste water = Average discharge of wastewater (m^3/sec)

D. Calculation of Pipe Dimensions

Pipes made of concrete are chosen in the planning of domestic wastewater networks because the material is not easy to react and has a large size. Calculations for the dimensions of concrete pipes, which have a manning coefficient (n) of 0.012 - 0.016. The results of the calculation of pipe dimensions can be seen in **Table 4.**

Table 4. Calculations to get pipe dimensions and values for V_{full}

Qmd	Qinf	Qpeak	Qdesign	n pipe	Diameter (mm)		Vfull	Qfull	Qd/Qf	d/D	Vp/Vfull	Clean Out and Manhole
(m^3/s)	(m^3/s)	(m^3/s)	m^3/s		Theoreticale (mm)	Market (mm)	m/s	m^3/s				
0,00017	0,00043	0,00036	0,00079	0,014	45,135	100	0,84	0,007	0,12	0,17	0,48	1
0,00138	0,00042	0,00290	0,00332	0,014	73,856	100	0,95	0,007	0.45	0,45	1,00	0
0,00281	0,00046	0,00583	0,00629	0,014	82,409	100	1,34	0,011	0,60	0,59	1,12	0
0,00318	0,00027	0,00659	0,00686	0,014	98,273	100	0,91	0,007	0,96	0,84	1,15	1
0,00321	0,00034	0,00662	0,00696	0,014	111,500	125	0,77	0,009	0,74	0,63	1,12	1
0,00323	0,00023	0,00667	0,00689	0,014	90,438	100	1,15	0,009	0,77	0,75	1,13	0
0,00325	0,00014	0,00670	0,00684	0,014	106,972	125	0,84	0,010	0,66	0,60	1,12	0
0,00329	0,00014	0,00674	0,00689	0,014	94,180	100	1,03	0,008	0,85	0,79	1,14	0
0,00343	0,00063	0,00697	0,00760	0,014	147,188	250	0,64	0,031	0,24	0,27	0,64	0
0,00345	0,00067	0,00700	0,00767	0,014	115,213	125	0,78	0,010	0,81	0,63	1,12	1
0,00450	0,00029	0,00907	0,00936	0,014	92,964	100	1,45	0,011	0,82	0,78	1,12	0
0,00663	0,00031	0,01318	0,01349	0,014	108,259	125	1,61	0,020	0,68	0,90	1,19	0
0,00679	0,00051	0,01335	0,01386	0,014	155,646	200	0,86	0,027	0,51	0,50	1,10	1
0,00896	0,00037	0,01741	0,01778	0,014	131,124	150	1,44	0,025	0,70	0,61	1,12	0
0,00896	0,00017	0,01741	0,01758	0,014	139,333	150	1,21	0,021	0,82	0,59	1,12	0
0,01793	0,00023	0,03204	0,03227	0,014	171,330	200	1,55	0,049	0,66	0,64	1,12	0

Annotation:

1. Qmd = Maximum discharge of wastewater in 1 day (m^3/second)
2. Qinf = Infiltration discharge (m^3/sec)
3. Qpeak = Peak discharge of wastewater (m^3/sec)
4. Q design = Total infiltration discharge and peak discharge (m^3/sec)
5. n Pipe = Manning coefficient (roughness of the pipe)
6. Vfull = Maximum speed (0.6-3 m^3/s)
7. Qfull = Maximum discharge (m^3/sec)
8. Qd/Qf = The ratio of the discharge of water needs per day to the maximum discharge
9. d/D = the ratio of the water level to the pipe diameter
10. Vp/Vfull = The ratio of the minimum speed to the maximum speed

The planning of the piping network from the calculations that have been obtained is seen in **Figure 5.**

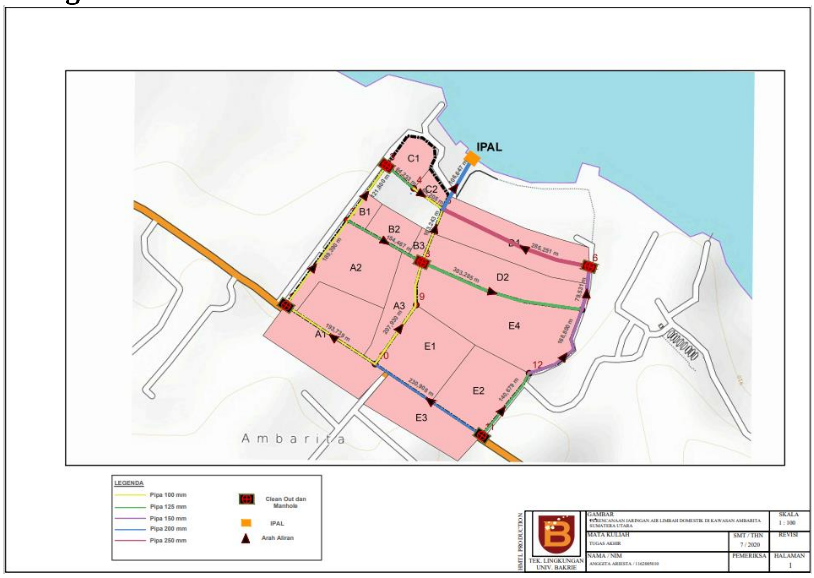

Figure 5.
Planning for Domestic Wastewater Piping Network in Ambarita Area

E. Sewerage with a Closed System

The distribution of domestic wastewater is carried out in a closed system, which aims to ensure that the distribution does not interfere with activities in the vicinity. The treated water or WWTP effluent will be flowed into the waters of Danu Toba, in accordance with the established quality standards, namely Peraturan Pemerintah Number 82 of 2001 class 2 (Presiden Republik Indonesia, 2001), which is designated for water recreation infrastructure / facilities, freshwater fish farming, animal husbandry, and irrigation.

F. Equipment and Types of Piping used

The equipment and types of piping that will be used for piping in the Ambarita area are manholes, clean out, and using concrete pipes.

G. Domestic Wastewater Treatment Plant Capacity

The quantity of wastewater to be treated at the wastewater treatment plant has been obtained from previous calculations, which is the amount of the average discharge of wastewater, the WWTP discharge is 0.06495 m^3/s and the WWTP capacity is obtained as follows.

$$\textbf{Daily WWTP capacity } (\mathbf{m^3/day}) = \sum \textbf{Qr of waste water}$$

$$= 0{,}06495 \ \frac{\mathrm{m3}}{\mathrm{s}} x \ 86400 \frac{second}{day}$$

$$= 5611{,}460 \frac{m3}{day}$$

The design that will be planned in determining the WWTP Layout in the Ambarita area can be seen in **Table 5.**

Table 5. Dimensions for Layout of Domestic Wastewater Treatment Plants

Average Discharge of Wastewater (Qr)		Volume of WWTP (m^3/day)	Dimention of WWTP (m)	Top View of the WWTP (m)	Land area of WWTP and available supporting buildings (m^2)	Land length (m)	Land width (m)
m^3/s	0,06495	8000	40x20x10	40x20	2000	50	40
m^3/s	5611,46						

The following is the WWTP layout based on the planning design which can be seen in **Figure 6.**

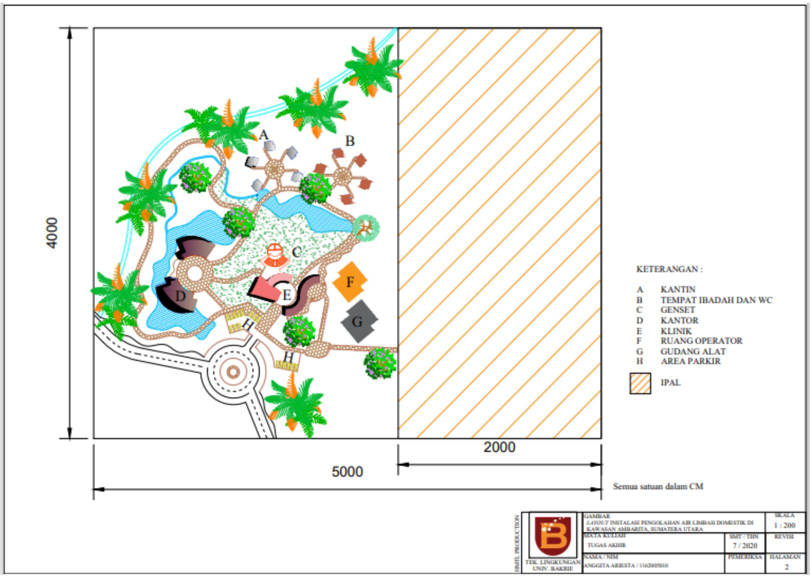

Figure 6.
WWTP Layout in Ambarita Area

H. Technology Recommendations for Domestic Wastewater Treatment Plants

The laboratory test results of the water quality of Lake Toba at Siallagan Harbor, which is located in Ambarita, can be seen at **Table 6.**

Table 6.
Water Sample Laboratory Test at Siallagan Port

No.	Parameter	Results	Quality Standard (PP No. 82 Tahun 2001 Kelas 2)	Unit	Information
1	COD	12,97	25	mg/l	Does not exceed
2	TSS	3,5	50	mg/l	Does not exceed
3	BOD	< 2	3	mg/l	Does not exceed
4	*Fecal Coli*	150	100	jml/100ml	Exceed
5	*Free Chlorine*	0,32	0,03	mg/l	Exceed
6	Total phosphate as P	0,0228	0,2	mg/l	Does not exceed

Source: The Environmental Service Laboratory in Pangururan, 2019

In the table above that the parameters that exceed class II quality standards Peraturan Pemeri Number 82 of 2001 (Presiden Republik Indonesia, 2001) concerning Water Quality Management and Water Pollution Control are fecal coliform and free chlorine. According to the characteristics of domestic wastewater, fecal coliform is included in biological characteristics, whereas, free chlorine is classified as organic chemical characteristics which usually comes from the use of detergents from hotels, households, and other public facilities. Biological processing technology is selected in the removal of these two parameters because of its lower cost than chemical processing and limited land availability. The choice of biological technology is also a request from the Samosir Regency Environmental Service. Several alternative technologies that are assessed based on the advantages and disadvantages can be seen in **Table 7.**

Table 7. Wastewater Treatment Alternative Assessment

Criteria	Activated Sludge	Biofilter	Trickling Filter	RBC[1]	ABR[1]
Land Availability	2	4	3	3	3
The suitability of treatment technology with the characteristics of wastewater	5	5	3	4	3
Removal efficiency	4	3	5	4	3
Investment Costs	4	3	2	2	2
Maintenance cost	4	5	4	2	2
Types of equipment and spare parts	5	3	3	3	3
Needs and uses of this type of energy	1	5	3	3	4
Ease of operation	5	4	3	3	3
Aesthetics	1	4	4	4	4
Total Value	**31**	**36**	**30**	**28**	**27**

Source : Said (2008) in Asmadi dan Suharno (2012) in (Nasoetion et al., 2017), [1]Hasil analisis

Based on the total value of technology selection with several assessment criteria, the biofilter was selected as an alternative technology to be used in domestic wastewater treatment in the Ambarita area. The biofilter that will be used in domestic wastewater treatment in the Ambarita area is anaerobic-aerobic biofilter, because the use of anaerobic processes, organic pollutants in wastewater will be converted into carbon dioxide gas and methane which is broken down by anaerobic bacteria and facultative bacteria without using energy. like an air blower, but the content of ammonia and hydrogen sulfide gas (H_2S) cannot be lost. So, if only anaerobic biofilter is used, only organic pollutants such as BOD, COD and TSS can be removed. Therefore, it is necessary to process an aerobic biofilter that can break down the remaining organic pollutants from the anaerobic process, where organic pollutants will be converted into CO_2 and H_2O (Kemenkes, 2011). An illustrative anaerobic-aerobic biofilter process can be seen in **Figure 7.**

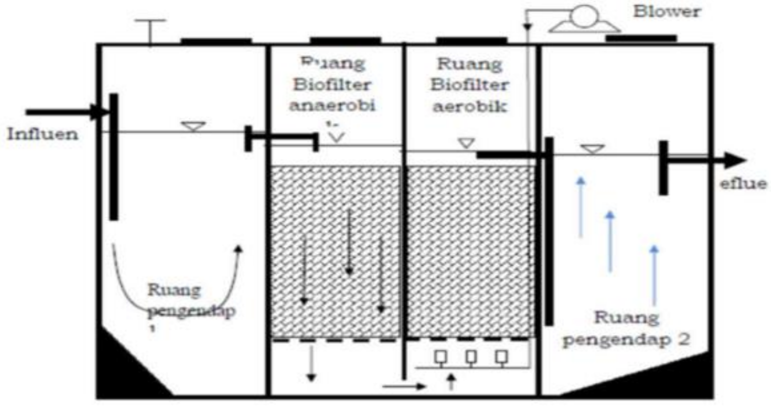

Figure 7.
Illustration of Anaerobic-Aerobic Biofilter Unit Model
Source : (Republik Indonesia, 2017)

In the provision of two parameters that exceed the quality standard of Peraturan Pemerintah Number 82 of 2001 (Presiden Republik Indonesia, 2001), namely *fecal coli* and free chlorine, the efficiency of the aerobic-anaerobic biofilter according to (Hariyani & Sarto, 2018).

1. Fecal coliform Removal

- Removal: 99,9% at peak hour discharge
- Initial concentration = 150 coliform/100 ml
- Final concentration = 150 – (150 x 99,9%) = 0,15 coliform/100ml, which is less than 100 coliform/100ml.

2. Free chlorine Removal
   The removal of free chlorine in domestic wastewater is more than 65% using an aerobic-anaerobic biofilter (Häggblom & Salkinoja-Salonen, 1991).
   - Initial concentration = 0,32 mg/l
   - Final concentration = 0,32 – (0,32 x 65%) = 0,112 mg/l
   - 0,112 mg/l still exceeds class 2 water quality standards stipulated in Peraturan Pemerinta Number 82 of 2001(Presiden Republik Indonesia, 2001), namely 0.03 mg / l. So, there needs to be an effort to reduce the free chlorine concentration. However, according to (Hasan, 2006) the chlorine content in fresh water naturally is 8.3 mg/liter. This shows that the reduction of free chlorine after deep processing using anaerobic-aerobic biofilter technology is still safe.

For the planned influent, the characteristics of domestic wastewater have key parameters, namely BOD, COD, and TSS. The average values of BOD, COD, and TSS for the characteristics of domestic wastewater are 220 mg/l, 500 mg/l, and 220 mg/l (Metcalf et al., 1991).

1. BOD Removal
   BOD removal in domestic wastewater using anaerobic-aerobic biofilter was 85% (Metcalf et al., 1991).
   - Initial concentration = 220 mg/l
   - Final concentration = 220 mg/l – (220 mg/l x 85%) = 33 mg/l
   - The BOD allowance exceeds the class 2 quality standard according to Peraturan Pemerintah Number 82 of 2001 (Presiden Republik Indonesia, 2001), which is 3 mg / l, however in the (Permen LHK, 2016) concerning Domestic Wastewater Quality Standards, the BOD value set is 30 mg / l where the effectiveness of the reduction has reached 91%.
2. COD Removal
   Removal of COD in domestic wastewater using anaerobic-aerobic biofilter is 85% (Metcalf et al., 1991).
   - Initial concentration = 500 mg/l
   - Final concentration = 500 mg/l – (500 mg/l x 85%) = 75 mg/l
   - The COD allowance still exceeds the quality standard of Class 2 Peraturan Pemerintah Number 82 of 2001 (Presiden Republik Indonesia, 2001), which is 25 mg / l with the effectiveness of the reduction is still 34%, however (Permen LHK, 2016) concerning Domestic Wastewater Quality Standards, the COD value set is 100 mg/l. This shows that the removal of COD with anaerobic-aerobic biofilter has met the quality standards of the Minister of Environment Regulation Number 68 of 2016.
3. TSS Removal
   80% removal of TSS in domestic wastewater using anaerobic-aerobic biofilter (Metcalf et al., 1991).
   - Initial concentration = 220 mg/l
   - Final concentration = 220 mg/l – (220 mg/l x 80%) = 44 mg/l
   - The TSS allowance has met the class 2 quality standard according to Peraturan Pemerintah Number 82 Year 2001 (Presiden Republik Indonesia, 2001) of 50 mg / l, but has not met the quality standard for the Minister of Environment

Regulation Number 68 Year 2016 concerning Domestic Wastewater Quality Standards with a TSS value set at 30 mg/l. Thus, the effectiveness of TSS reduction is still 69%.

CONCLUSIONS

The conclusions from the above discussion are as follows:

1. Quantity and quality of domestic wastewater in the Ambarita area:
   a. The total discharge of wastewater in the planning area is = 0.06495 m^3/sec
   b. The quality of the waters of Lake Toba which indicates pollution from domestic wastewater is COD 12.97 mg/l, TSS 3.5 mg/l, BOD <2 mg/l, *fecal coli* 150 coliform/100ml, Free Chlorine 0.32 mg/l, and total phosphate as P 0.0228 mg/l. The only parameters that exceed the quality standard are free chlorine and *fecal coli*.
2. The planned waste water distribution system is an Ambarita service area distribution system in the MasterPlan draft that focuses on congested activities. The pipe used is a concrete pipe because is strong enough for the distribution of domestic wastewater. The diameter used for pipes made of concrete is 100 mm, 125 mm, 150 mm, 200 mm, and 250 mm and it will have 5 manholes.

3. The total length of pipe required for each diameter of the commonly pipe used in the market, 100 mm = 1010 m; 125 mm = 663 m; 150 mm = 248 m; 200 mm = 338 m and for 250 mm = 285 m.

4. Wastewater treatment plants selected to use anaerobic-aerobic biofilter with effectiveness in reducing fecal coli and free chlorine by 99.9% and 65%. Although, the free chlorine allowance has not met the quality standards of Peraturan Pemerintah Number 82 Year 2001 class 2 (Presiden Republik Indonesia, 2001), the use of this anaerobic-aerobic biofilter is the most effective alternative technology.

5. Reduction of the key parameters of domestic wastewater, namely BOD, COD and TSS with allowances of 85%, 85% and 80% in using anaerobic-aerobic biofilter technology.

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