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Assessing the Performance of Urban Drainage Networks in Settlements as an Impact of Urbanization

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Abstract. The increase in the number of residents in the city area due to urbanization and the rapid growth of the city's population itself causes many environmental problems, one of which is flooding. The drainage network becomes a vital infrastructure in the development of the city to be free from flood problems, especially in dense areas such as settlements or housing, which are increasingly densely populated every year. The purpose of this study is to examine whether the drainage network in the Berlian Simpati Blorep housing estate in the middle of Merauke City is able to accommodate flood discharge for the next few years. The methods used in this study are quantitative and qualitative, based on the results of the analysis. The results showed that with an existing channel capacity (Qs) of 0.097 m3/sec < the planned flood calculation discharge (Qr) of 13.657 m3/sec, the existing channel capacity was not able to function effectively with a 5-year recurrence period.

Keywords: urban drainage networks, settlements, urbanization.

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1 Introduction

Drainage networks are vital infrastructure that is very important in an urban area [1]. A good drainage network can free cities from flooding problems caused by relatively fast population growth and reduced green land [2]. In 2022, the population of Merauke City will be 102,689 people, representing 44.32% of the total population of Merauke Regency. The population growth rate in 2010–2020 was 0.01%, and in 2021–2022, it was 0.33% [3], meaning that the population growth rate in the last year has increased very significantly compared to the previous ten years.

The rapid rate of population growth is influenced by urbanization and population growth in the city [4]. Merauke City, which is the center of economy, industry, and education, encourages people from outside the city to urbanize. Rapid urbanization has an impact on population density that cannot be contained in both settlements and housing. Dense settlements often cause environmental problems if adequate drainage networks are not provided, one of which is flooding [5]. Flooding is always a problem during the rainy season and is further exacerbated because the city of Merauke is in a swampy area that tends to be sloping so that water is difficult to flow by gravity [6].

To overcome the above problems, it is necessary to conduct a study or analysis of the performance of drainage networks in residential areas in Merauke City. Research on drainage networks has been done before, but so far it has been limited to the performance of primary drainage channels that lead directly to the sea or river [7], [8]. This research will focus on the performance of secondary drainage networks in residential areas.

The purpose of this study is to identify the performance of drainage networks in Merauke city settlements so as to prevent flooding or other environmental problems.

2 Research Methods

2.1 Research sites

This research is located in Simpati Blorep Diamond Housing, Merauke, with a geographical location at 08°30'08" S and 140°25'20" E. A map and research location can be seen in Figure 1.

Fig. 1. Drainage network research location

2.2 Data collection technique

The data collected are primary data and secondary data [9]. Primary data is data obtained from direct observations in the field, including cross-sectional measurements and channel network conditions [10].

Secondary data is supporting data obtained from various sources in the form of journals and books related to this research. In this study, more refers to or is influenced by secondary data. These numbers include the following.

- Map of the drainage canal network
- Rainfall data

2.3 Data analysis

The research stages consist of hydrological analysis and hydraulic analysis. Hydrological analysis was carried out to determine the rain plan for the 5th and 10th periods, after which the large flood discharge plan was obtained.

2.3.1 Hydrological Analysis

Calculating Rain Plans with the Log Pearson Type III Method [7], [11]

$$Log\,X_T = \overline{Log\,X} + K_T \times S\,Log\,X \qquad (1)$$

Calculating the values of the skewness coefficient (Cs) and curtosis coefficient (Ck) [12], [13]

$$C_S = \frac{n.\sum_{i=1}^{n}(X_i - X)^2}{(n-1).(n-2).S^3} \qquad (2)$$

$$C_k = \frac{n.\sum_{i=1}^{n}(X_i - X)^4}{(n-1).(n-2).(n-3).S^4} \qquad (3)$$

Calculating probability by using the Weibull e Research Location 14]

$$P = \frac{m}{n+1}\,.\,100\% \qquad (4)$$

Calculating rain intensity empirically using the Mononobe method [5], [15]

$$I = \frac{R_{24}}{t}\left(\frac{t}{Tc}\right)^{\frac{2}{3}} \qquad (5)$$

Calculating plan flood discharge with the Rational Method [16], [17]

$$Q_T = 0{,}278\,.\,C\,.\,I\,.\,A \qquad (6)$$

2.3.2 Hydraulic Analysis

Rainwater runoff in an area must be directly drained so as not to cause inundation or flooding[18]. The flow rate of the channel is affected by the shape, roughness, and slope of the channel. Therefore, the storage capacity is determined based on the amount of rainwater discharge and dirty water discharge.

Calculating the capacity of the existing channel using a quadrangular cross-sectional formula [19].

$$A = b \times h \quad (7)$$

$$P = b + 2h \quad (8)$$

$$R = \frac{A}{P} \quad (8)$$

Calculating the discharge capacity of a channel plan using the Manning equation [17].

$$V = \frac{1}{n} . R^{2/3} . S^{1/2} \quad (9)$$

$$Q = \frac{1}{n} . A . R^{2/3} . S^{1/2} \quad (10)$$

The estimated amount of wastewater flow discharge is divided into three parts [17], namely:
- Domestic wastewater,
- Industrial and commercial wastewater,
- Surface water infiltration (rain) and groundwater.

To find out the need for clean water in the service area, it is necessary to know the standards used and the facilities to be served for both domestic and non-domestic needs [20], which can be seen in Table 1.

Tabel 1. Individual domestic wastewater quality standards

Parameter	Unit	Maximum Rate *
pH	mg/L	6 – 9
BOD	mg/L	30
COD	mg/L	100
TSS	mg/L	30
Oil and fat	mg/L	55
Ammonia	mg/L	10
Total Coliforms	Amount /100mL	3000
Debit	L/Person /Day	100

Dirty water discharge can be calculated using the following equation [20]:

$$Q_{ak} = Q_{kep} . P_n \quad (11)$$

3 Results and Discussion

3.1 Rainfall Data Analysis

The rainfall data obtained is monthly rainfall data for a period of 10 years (2011–2020). The data is then taken as analysis data, which can be seen in Table 2.

Table 2. Monthly maximum rainfall data

No	Year	Maximum Rainfall (mm)	Month
1	2011	530,30	March
2	2012	525,90	March
3	2013	575,90	January
4	2014	482,70	April
5	2015	327,00	February
6	2016	428,90	February
7	2017	480,80	April
8	2018	356,60	March
9	2019	643,00	March
10	2020	336,10	February

The monthly maximum rainfall data is then sorted from small to large to calculate the skewness coefficient (Cs) and determine the probability factor (K) value. The values of Cs and K are obtained as shown in Table 3.

Table 3. The value of the skewness coefficient (Cs) and K

Return Period (T)	Skewness coefficient (Cs)	K
2	-0,2798	0,0466
5	-0,2798	0,8524
10	-0,2798	1,2476
20	-0,2798	1,5834

Based on the data above, planned rainfall for 2-year, 5-year, 10-year, and 20-year periods is calculated using the Log Pearson Type III method as follows:

$X_2 = 2{,}6605 + (0{,}0466 \times 0{,}1013) = 2{,}6652$
$X_2 = 10^{2{,}6652} = 462{,}59$ mm

$X_5 = 2{,}6605 + (0{,}8524 \times 0{,}1013) = 2{,}7468$
$X_5 = 10^{2{,}7468} = 558{,}21$ mm

$X_{10} = 2{,}6605 + (1{,}2476 \times 0{,}1013) = 2{,}7868$
$X_{10} = 10^{2{,}7868} = 612{,}07$ mm

$X_{20} = 2{,}6605 + (1{,}5834 \times 0{,}1013) = 2{,}8208$
$X_{20} = 10^{2{,}8208} = 661{,}91$ mm

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A summary of the calculation results can be seen in Table 4.

Table 4. Rain forecast plan period T (year)

Return Period (T)	Log $\bar{X}$	Cs	Sd	K	Log XT	X (mm)
2	2,6605	-0,2798	0,1013	0,0466	2,6652	462,59
5	2,6605	-0,2798	0,1013	0,8524	2,7468	558,21
10	2,6605	-0,2798	0,1013	1,2476	2,7868	612,07
20	2,6605	-0,2798	0,1013	1,5834	2,8208	661,91

The rainfall intensity of the next plan can be calculated using the Mononobe formula. The first step calculates the average t-hour rain with a 5-hour rain forecast (t).

$$T = 1 \text{ hour, } R_t = \frac{R_{24}}{5} \times \left[\frac{5}{1}\right]^{\frac{2}{3}} = 0{,}585 \times R_{24}$$

$$T = 2 \text{ hour, } R_t = \frac{R_{24}}{5} \times \left[\frac{5}{2}\right]^{\frac{2}{3}} = 0{,}368 \times R_{24}$$

$$T = 3 \text{ hour, } R_t = \frac{R_{24}}{5} \times \left[\frac{5}{3}\right]^{\frac{2}{3}} = 0{,}281 \times R_{24}$$

$$T = 4 \text{ hour, } R_t = \frac{R_{24}}{5} \times \left[\frac{5}{4}\right]^{\frac{2}{3}} = 0{,}232 \times R_{24}$$

Next calculate the rainfall at the t-hour as follows:

$T = 1$ *hour, so* $Rt = 1 \times (0{,}585 \times R_{24}) - (1 \times 1) = 0{,}585 \times R_{24} = 58{,}8\%$
$T = 2$ *hour, so* $Rt = 2 \times (0{,}368 \times R_{24}) - (2 \times 1) = 0{,}152 \times R_{24} = 15{,}2\%$
$T = 3$ *hour, so* $Rt = 3 \times (0{,}281 \times R_{24}) - (3 \times 1) = 0{,}107 \times R_{24} = 10{,}7\%$
$T = 4$ *hour, so* $Rt = 4 \times (0{,}232 \times R_{24}) - (4 \times 1) = 0{,}085 \times R_{24} = 8{,}5\%$
$T = 5$ *hour, so* $Rt = 5 \times (0{,}200 \times R_{24}) - (5 \times 1) = 0{,}072 \times R_{24} = 7{,}2\%$

Details of the calculation results are shown in Table 5.

Table 5. Recapitulation of design rainfall intensity calculations

Rain Duration (hour)	Ratio (%)	Hourly Rainfall (mm)					
		2-year	5-year	10-year	20-year	50-year	100-year
1	58,5	270,615	326,553	358,061	387,217	417,099	438,832
2	15,2	70,314	84,848	93,035	100,610	108,374	114,021
3	10,7	49,497	59,728	65,491	70,824	76,290	80,265
4	8,5	39,320	47,448	52,026	56,262	60,604	63,762
5	7,2	33,306	40,191	44,069	47,658	51,335	54,010
Total Rainfall		463,052	558,768	612,682	662,571	713,702	750,890

The table above shows total rain values for a 2-year birthday period of 463,052 mm, a 5-year birthday period of 558,768 mm, a 10-year birthday period of 612,682 mm, a 20-year birthday period of 662,571 mm, a 50-year birthday period of 713,702 mm, and a 100-year repeat period of 750,890 mm.

3.2 Calculation of the Existing Discharge of Drainage Channels

The existing channel that has been measured in the field is symbolized by C (channel). The dimensions of existing channels can be seen in Table 6.

Table 6. Existing channel data

No	Section	Channel Existing Dimensions	
		B (m)	H (m)
1	C1	0,8	0,6
2	C2	0,8	0,6

3	C3	0,8	0,6
4	C4	0,8	0,55
5	C5	0,74	0,62
6	C6	0,82	0,66
7	C7	0,79	0,69
8	C8	0,79	0,67
9	C9	0,71	0,6
10	C10	0,78	0,61
11	C11	0,75	0,6
12	C12	0,65	0,65
13	C13	0,74	1,3
14	C14	0,77	0,97

Description: C1 = Channel 1, C2 = Channel 2, etc.

From the data above, the existing channel capacity can be calculated by calculating in advance the average flow speed at a distance of 2 meters as follows:

- Flow speed with a travel time of 17 seconds

$$v_1 = \frac{2}{17} = 0{,}12 \text{ m/s}$$

- Flow speed with a travel time of 13 seconds

$$v = \frac{2}{13} = 0{,}15 \text{ m/s}$$

- Flow speed with a travel time of 15 seconds

$$v = \frac{2}{15} = 0{,}13 \text{ m/s}$$

- Average flow speed

$$V_{average} = \frac{0{,}12 + 0{,}15 + 0{,}13}{3} = 0{,}13 \text{ m/s}$$

The average flow velocity is used to calculate the existing capacity of the channel using a quadrangular cross-sectional formula, as illustrated in Figure 2.

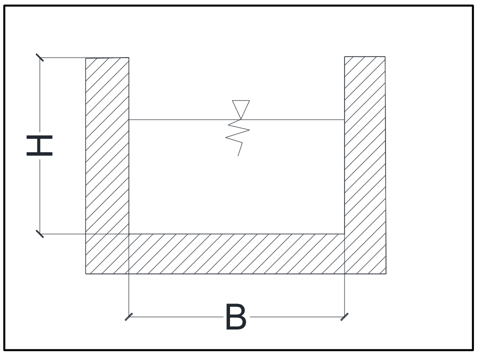

Fig. 2. Square channel cross section

The results of the analysis can be seen in Table 7.

Table 7. Recapitulation of the existing discharge of each drainage channel

No	Section	Channel Existing Dimensions		A (m^2)	P (m)	R (m)	Qs (m^3/s)
		B (m)	H (m)				
1	C1	0,8	0,6	0,48	2	0,24	0,062
2	C2	0,8	0,6	0,48	2	0,24	0,062
3	C3	0,8	0,6	0,48	2	0,24	0,062
4	C4	0,8	0,55	0,44	1,9	0,23	0,057
5	C5	0,74	0,62	0,46	1,98	0,23	0,060
6	C6	0,82	0,66	0,54	2,14	0,25	0,070
7	C7	0,79	0,69	0,55	2,17	0,25	0,071
8	C8	0,79	0,67	0,53	2,13	0,25	0,069
9	C9	0,71	0,6	0,43	1,91	0,22	0,055
10	C10	0,78	0,61	0,48	2	0,24	0,062
11	C11	0,75	0,6	0,45	1,95	0,23	0,059
12	C12	0,65	0,65	0,42	1,95	0,22	0,055

13	C13	0,74	1,3	0,96	3,34	0,29	0,125
14	C14	0,77	0,97	0,75	2,71	0,28	0,097

3.3 Calculation of Flood Discharge for Drainage Channel Plans

The calculation of flood discharge was obtained based on the analysis of maximum rainfall, and then field reviews and measurements obtained a catchment area of 0.447 km2.

Plan flood discharge is calculated using the Rational Method with re-periods (T) of 5 and 10 years. The drainage coefficient is taken at a value of 0.5 for residential areas. The slope of the channel is taken at 3% based on the condition of the slope of the channel for dirty water.

The discharge plan for the channel capacity plan also needs to take dirty water discharge into account. Dirty water discharge is calculated by taking into account the number of people living in settlements and the amount of domestic wastewater for residents, which is 100 liters per person per day. The results of the planned discharge analysis of each channel can be seen in Table 8.

Table 8. Calculation of flood discharge planned for 5-year and 10-year periods

No	Section	Planned flood discharge (m^3/s)	
		5-year	10-year
1	C1	1,329	1,457
2	C2	1,145	1,256
3	C3	11,190	12,270
4	C4	0,990	1,086
5	C5	1,089	1,195
6	C6	1,184	1,299
7	C7	1,219	1,336
8	C8	1,285	1,409
9	C9	1,216	1,333
10	C10	1,201	1,317
11	C11	1,091	1,197
12	C12	1,031	1,130
13	C13	0,884	0,970
14	C14	13,657	14,982

From the calculation results in the table above, the amount of planned rain discharge in the 5-year period (Q5) was 13,657 m3/sec and in the 10-year period (Q10) was 14,982 m3/sec.

Based on the results of the calculation of flood discharge for the 5-year anniversary period and the existing discharge, a review of both was carried out, as presented in the following table.

Table 9. Recapitulation of the existing discharge of each drainage channel

No	Section	Existing channel discharge (m3/sec)	Planned flood discharge (m3/sec)	Information
1	C1	0,062	1,329	Unable
2	C2	0,062	1,143	Unable
3	C3	0,062	11,190	Unable
4	C4	0,057	0,990	Unable
5	C5	0,060	1,089	Unable
6	C6	0,070	1,184	Unable
7	C7	0,071	1,219	Unable
8	C8	0,069	1,285	Unable
9	C9	0,055	1,216	Unable
10	C10	0,062	1,201	Unable

11	C11	0,059	1,091	Unable
12	C12	0,055	1,031	Unable
13	C13	0,125	0,884	Unable
14	C14	0,097	13,657	Unable

The analysis results in Table 8 show that the existing channel capacity (Qs) is 0.097 m3/sec < the planned flood calculation discharge (Qr) is 13.657 m3/sec, so the existing channel capacity is not able to function effectively with a 5-year recurrence period.

4 Conclusion

Based on the results of the analysis over a 5-year period, it was found that the existing channel capacity (Qs) was 0.097 m3/sec < the planned flood calculation discharge (Qr) was 13.657 m3/sec. This condition shows that the existing channel capacity of the drainage network in the Berlian Simpati Blorep settlement is ineffective and needs to be re-planned to be free from inundation.

Replan by increasing the channel dimension, i.e., by increasing the channel height. This step is more likely than increasing the width of the channel because some of the channels on the right and left are buildings except for the channel that is a ditch with a road.

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