Evaluation on Merauke Drainage System in Overcoming Flood-Prone Areas

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Abstract

Merauke city suffers flooding in almost every rainy season, which endangers the environment and disturbs people's activities. One of the worst floods happened in 2019, where flooding damaged vegetables and crops, impacted the city's economy, and later caused economic inflation. Therefore, a study evaluating the drainage system in Merauke city is needed to overcome this issue. This study was conducted through a field survey and literature study with data that varies from hydrological, elevation, and hydraulics data. Hydrological data includes yearly rainfall data, and hydraulics data consists of the drainage network and designs. This evaluation will then result in the normalization of a drainage system. This study found that the capacity of the current drainage system is 10.815 m^3/s, where this drainage capacity cannot accommodate the amount of flood discharge plan. Also, the existing drainage system is missing a component that was not built in the first place, affecting the drainage's capacity. Therefore, a normalization of the drainage system is needed to avoid flooding.

Key words: evaluation, drainage system, Merauke

1. Introduction

Merauke city is a small town located in the southern part of Papua Island and the most east of Indonesia, often called *"Kota Rusa"*, which means "The Deer Town". As the population and infrastructure are snowballing in Merauke, the local government has proposed Merauke city as the capital of South Papua Province, leading four other regencies: Merauke, Mappi, Asmat, and Boven Digoel. This matter must be accompanied by improvements in various aspects that can support a more systematic and more organized city for Merauke as a decent capital city for a new province of South Papua.

Merauke regency *(Kabupaten)* is located at 137° - 141° east longitude and 5° - 9° south latitude. With a total area of 46,719.63 km^2, or about 14.67% of the total area of Papua Province, Merauke is the largest regency in Papua. Merauke regency has 20 districts *(Kecamatan)*. Water areas in Merauke regency reach approximately 5,089.71 km^2 [1].

Merauke city typically has a low altitude and is surrounded by the Maro river in the north and the Arafura Sea in its west and south. With the condition of being surrounded by water, the problem commonly encountered in Merauke city is flooding. This event occurs pretty often in specific areas, later called flood-prone areas. The local government has already tried its best to overcome this issue, but the result was not satisfying. Recently, Merauke suffered yet another flood in 2019, and it was even worse than before, causing economic inflation in the price of vegetables and other food supplies [2].

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From Kumparan News March 2019, “The Meteorology, Climatology, and Geophysics Agency (BMKG) of Mopah Merauke Airport Station estimates that moderate-intensity rain will continue for the next three days. BMKG’s Weather Forecast Section on Kumparan news explained that the condition of moderate to heavy rain in Merauke city since yesterday was caused by the potential for tropical cyclone seeds to the south of Papua New Guinea (PNG)” [3].

Flooding is a significant issue in Merauke city, and solving this problem will need a deeper study on evaluating the current drainage system. This study will focus on analyzing the root cause of the problem of flooding in Merauke city so that all the areas that are often flooded in rainy seasons (flood-prone areas) can be overcome. Hopefully, this study could help to maximize the effort to fix the problem.

1.1 Research study location

This study was conducted in Merauke Regency located in southern part of Papua Island. This study focuses on urban areas which located in the heart of the regency: Merauke city, as marked with red circle in Fig. 1.

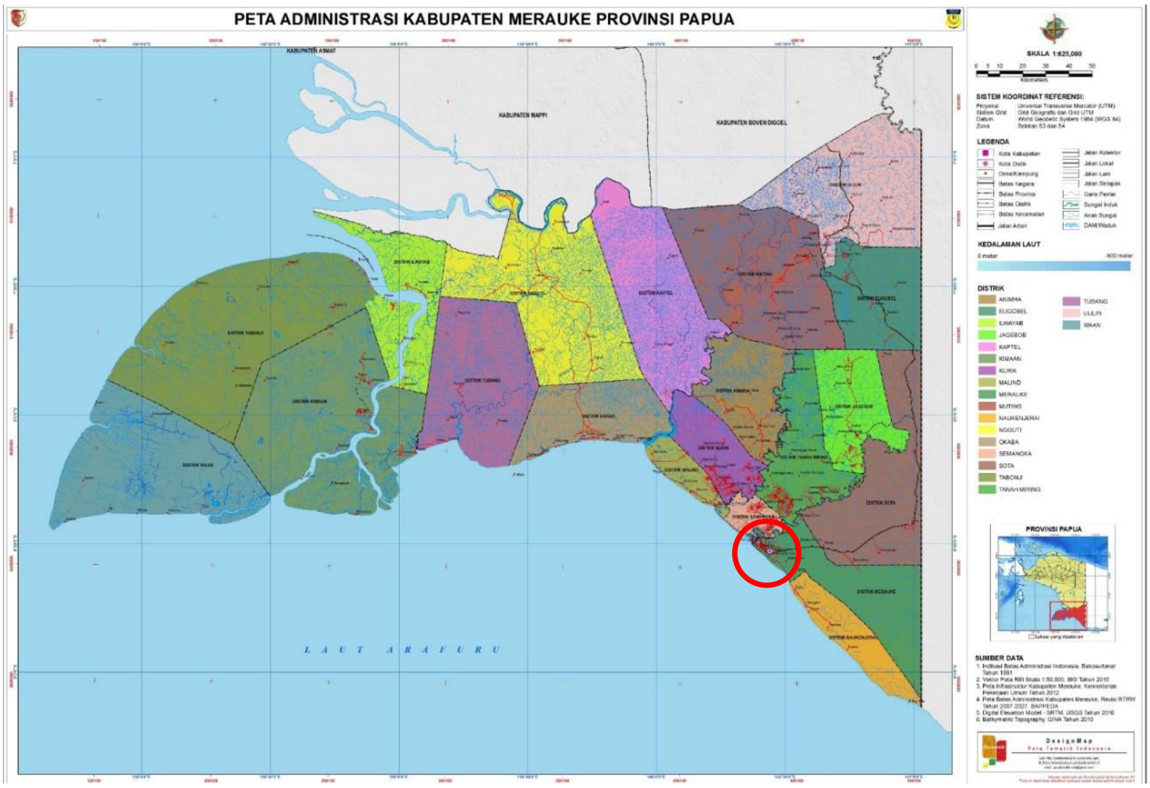

Fig. 1 Merauke regency administrative map [4]

1.2 Merauke topographical condition

According to Merauke's Regional Development Planning, Research and Development Agency or *Bappeda Kabupaten Merauke*, Merauke's topographical conditions are covered mainly by swampy land with low altitudes, some of which are below sea level. Merauke generally has low land, a steepness rate of 0-8%, and swampy coastal areas inundated [4].

Most areas are located at altitudes 0–60 meters above sea level. The flat areas are primarily in the southern part and center of the regency. This area is the center of the population which started the land use business for cultivation activities and the concentration of residential settlements [4].

1.3 Merauke demographical condition impact on land subsidence

Merauke is a small city with a small population and low tourism destinations. Most of the people that came to Merauke were either for family or business purposes. On their official website, the Indonesian Ministry of Civil Affairs stated that on the 31st of December 2021, the total population of Merauke city was 103,641 people, with the population growth rate in 2016-2018 only 2%. With this low population growth in Merauke city, land subsidence is not something that could be a problem in the next ten years. Also, there are no high-rise buildings, the tallest building in Merauke city was only a five-story hotel that reached less than 20 m above the ground [5].

2. Study Plan

All the data collected for this study has to be obtained from a reliable source, because not many academics have discussed this flood issue in Merauke city. Therefore, the most reliable data would be directly from the local government agency. This study will take the rainfall data from the nearest Meteorology Station of BMKG in Mopah Airport. The current drainage system data will be taken from Merauke Public Works and Spatial Planning Service.

This study will start with a hydrological analysis which requires daily rainfall data for at least 10 years back for the primary drainage system according to The National Standard of SNI-2415-2016 [6]. The rainfall data can be obtained from a local meteorology station and later analyzed through four distribution methods; Gumbel, Normal, Log Normal, and Log Pearson Type III. To determine which distribution methods are eligible to be used in the study, the calculation of those four methods will be tested with the Chi-Square and Smirnov Kolmogorov tests.

After the hydrological analysis has been done, the study will continue analyzing the amount of flood discharge plans. Exploring the flood discharge plan requires calculating the concentration-time (Kirpich formula) and rainfall intensity (Monobe formula) from the previously chosen rainfall distribution method.

When the amount of flood discharge plan has been acquired, compare it with existing drainage capacity; thus began, the hydraulics analysis. In this analysis, the formula used is from Manning to calculate the flood discharge of the existing drainage capacity. By comparing the flood discharge plan and existing drainage capacity, conclude whether the current drainage system can accommodate the amount of rainwater flood discharge plan or not. The study plan is divided to several steps as shown in Fig. 2.

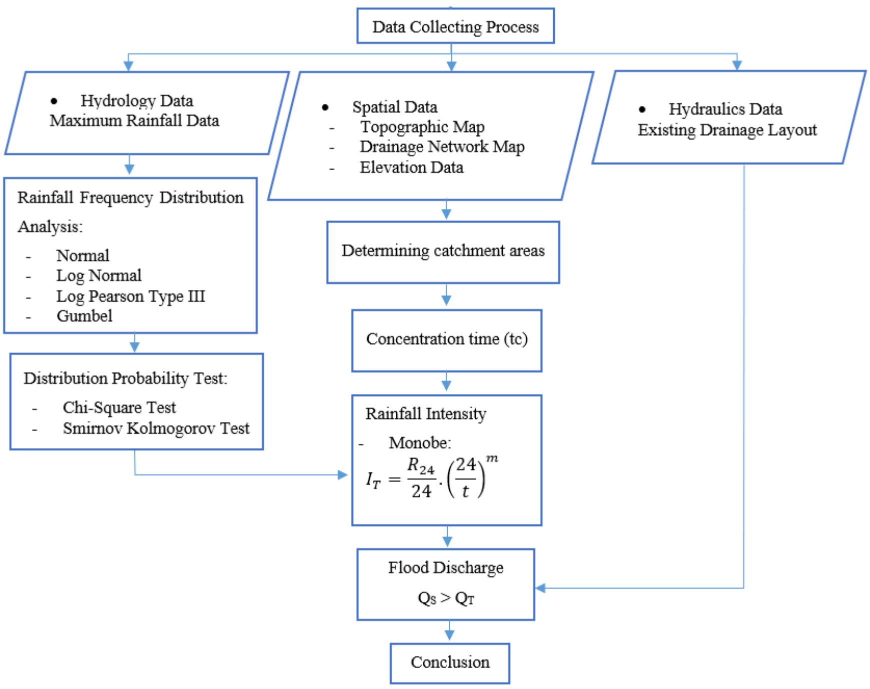

Fig. 2 Study plan

3. Results and Discussions

3.1 Hydrological analysis

The closest rainfall station or meteorology station is near the local airport called "Stasiun Meteorologi Mopah Merauke" which covers the whole city of Merauke. The rainfall data used in this study was taken from 2000-2020 for 21 data or 21 years (Table 1).

Table 1 Merauke rainfall data 2000-2020 from meteorology station of Mopah Merauke

No	Year	Max Rainfall (mm)
1	2000	80.00
2	2001	75.80
3	2002	103.20
4	2003	211.30
5	2004	117.10
6	2005	150.00
7	2006	141.20
8	2007	110.50
9	2008	89.50
10	2009	104.30
11	2010	198.60

No	Year	Max Rainfall (mm)
12	2011	124.20
13	2012	89.20
14	2013	281.60
15	2014	89.60
16	2015	79.10
17	2016	94.40
18	2017	121.80
19	2018	67.20
20	2019	194.80
21	2020	115.90

The rainfall data above will be carried out to four different distribution types for different year plans [6]. Table 2 below is the recap of the calculation.

Table 2 Rainfall distribution calculation recap.

Year Period	Rainfall Frequency			
	Log Normal	Normal	Log Pearson III	Gumbel
2	116.76	125.68	111.09	117.70
5	160.44	171.41	156.75	175.32
10	189.44	195.31	193.37	213.51
25	226.15	220.79	247.82	261.76

As already calculated the Rainfall Frequency using four different distribution methods, the next step will be to test those four methods and determine which one is the most eligible and can be used for further analysis. The tests that will be done are: Chi-Square and Smirnov Kolmogorov tests (Table 3 and Table 4).

Table 3 Chi-Square test calculation recap

	X^2		X^2cr	Status
Log Normal	1.00	<	7.815	Acceptable
Normal	12.43	<	7.815	Inacceptable
Gumbel	7.29	<	7.815	Acceptable
Log Pearson III	13.57	<	7.815	Inacceptable

Table 4 Smirnov Kolmogorov test calculation recap

	ΔP		ΔPcr	Status
Log Normal	0.103	<	0.290	Acceptable
Normal	0.178	<	0.290	Acceptable
Gumbel	0.925	<	0.290	Inacceptable
Log Pearson III	0.889	<	0.290	Inacceptable

After all the test has been done to all four rainfall distributions, the result of the test left out only Log Normal Distribution that pass both tests Chi-Square and Smirnov Kolmogorov. This distribution method will later be use in this study. In order to make it easy to analyze, the drainage banks will be divided into several catchment areas as seen in Fig. 3.

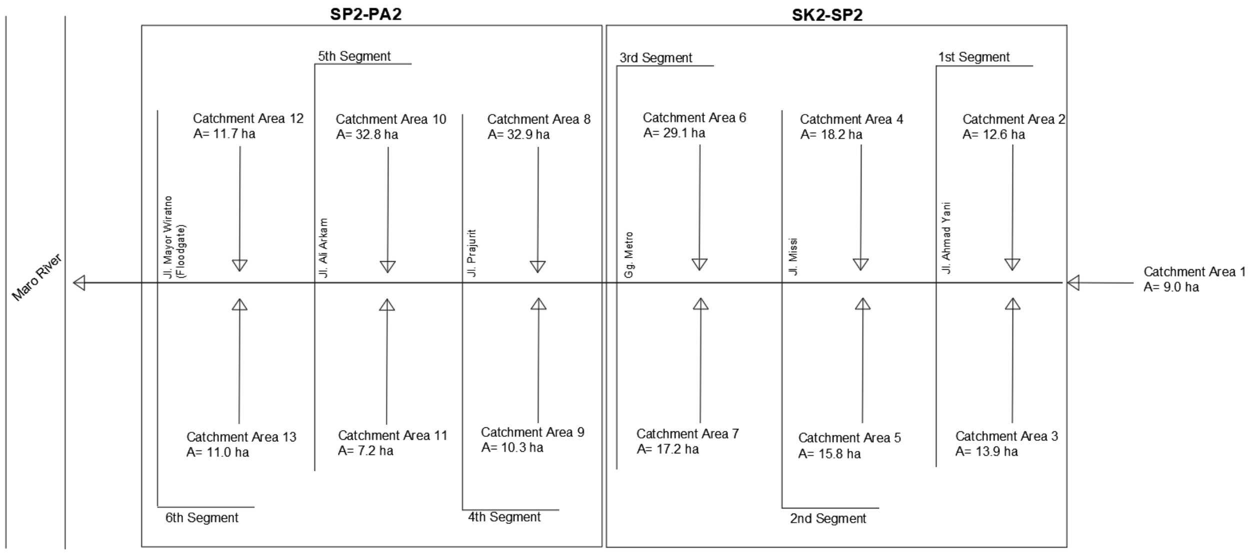

Fig. 3 Drainage system scheme

To find out how much rainwater that is collected inside the drainage (Table 5), will be using the Rational method:

$$Q = (0.00278) \times C \times I \times A$$

- Determining C

  Since the density of Merauke population is low, assume that C = 0.48 [6]

- Determining I

  Rainfall intensity will be calculated from the Monobe equation: $I_T = \frac{R_{24}}{24} \times \left(\frac{24}{Tc}\right)^{\frac{2}{3}}$

  The following is an example of the flood discharge plan calculation on the first catchment area:

$$Q = (0.00278) \times 0.48 \times 83.648 \times 9.05 = 1.0106\ m^3/s$$

Table 5 Flood discharge plan on each catchment areas

Catchment Area			Q (m^3/s)			
			2	5	10	25
SK2-SP2	1st Segment	1	1.0106	1.3887	1.6396	1.9573
		2	0.9201	1.2643	1.4928	1.7821
		3	0.9882	1.3579	1.6033	1.9140
	2nd Segment	4	1.1660	1.6021	1.8917	2.2583
		5	1.1123	1.5284	1.8046	2.1544
	3rd Segment	6	1.5577	2.1405	2.5273	3.0170
		7	1.0868	1.4933	1.7631	2.1048
SP2-PA2	4th Segment	8	0.9850	1.3534	1.5980	1.9077
		9	0.3330	0.4575	0.5402	0.6449
	5th Segment	10	1.3996	1.9231	2.2707	2.7107
		11	0.3278	0.4504	0.5318	0.6349
	6th Segment	12	0.6312	0.8673	1.0241	1.2225
		13	0.6361	0.8740	1.0319	1.2319

Since already obtained each catchment area's flood discharge plan, classify it into several segments according to where the rainwater will be contained later. The first three catchment areas ended up in the same drainage, so they can be classified into one segment. The following two catchment areas share the same drainage, so they can be classified into one segment, and the list goes on to the downstream. The segment classifications are also based on the data taken from Merauke Public Works and Spatial Planning Service.

Meanwhile, SK2-SP2 is the name of the drainage and indicates the type of drainage, which means it is a secondary drainage continuing to primary drainage and SP2-PA2 means a primary drainage continuing to the disposal. The name of the drainage was also taken from the data source.

The aim of classifying the catchment area is because the water in the downstream channels was not just containing from catchment areas but also from the previous drainage segments. Therefore, the flood discharge plan for segment 2 is the total amount of water from catchment areas 4 and 5, and the flood discharge plan from segment 1 and so on. Below is the example of the flood discharge calculation on the second segment in 2 years period plan: $Q_{S2} = Q_{CA4} + Q_{CA5} + Q_{S1} = Q_{CA4} + Q_{CA5} + (Q_{CA1} + Q_{CA2} + Q_{CA3}) = 1.1660 + 1.1123 + (1.0106 + 0.9201 + 0.9882) = 5.1972\ m^3/s$. The Flood discharge plan on segment areas as seen in Table 6.

Table 6 Flood discharge plan on segment areas

Segment Area		Q (m^3/s)			
		2	5	10	25
SK2-SP2	1	2.9189	4.0109	4.7357	5.6534
	2	5.1972	7.1414	8.4319	10.0660
	3	7.8417	10.7752	12.7223	15.1879
SP2-PA2	4	9.1596	12.5861	14.8605	17.7405
	5	10.8870	14.9597	17.6630	21.0861
	6	12.1543	16.7010	19.7190	23.5406

3.2 Hydraulics analysis

Hydraulics analysis aims to evaluate the channel capacity with a 10-year planned flood discharge for the primary channel. Full bank capacity is the amount of flood discharge in the channel according to the conditions in the field. This calculation is needed to determine how much the channel can accommodate rainwater runoffs. The existing drainage data taken from Merauke Public Works and Spatial Planning Service as shown in Fig. 4 and Fig. 5.

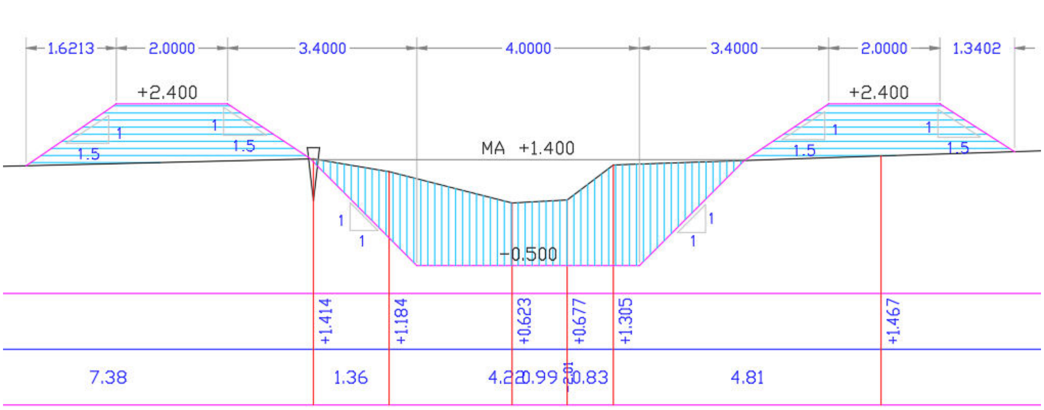

Fig. 4 Existing drainage SK2-SP2

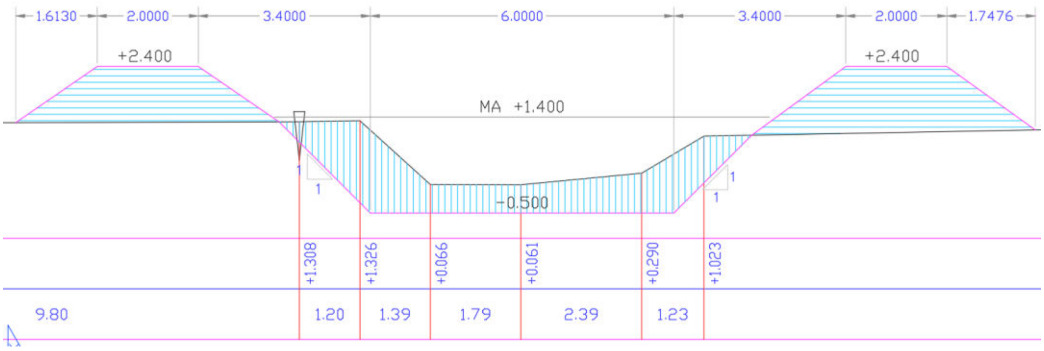

Fig. 5 Existing drainage SP2-PA2

Where:

- The black line indicates real condition of the land.
- Purple line indicates the planning of the drainage.
- Vertical blue stripes indicate the amount of soil that will be dug out.
- Horizontal blue stripes indicate the amount of soil that will be added for safety parameter.

This is the design that planned before the drainage was build. Since the drainage has already been built, the black line which indicates the land condition before the drainage was build, can be ignored. From two figures above it can be seen that the only thing that differentiate the first and second figure is the width on the bottom of the channel. Therefore, it can be concluded that:

- b1= 4 m
- b2= 6 m
- h = 1.4 + 0.5 = 1.9 m
- m = 1

Average velocity formula on channel calculation dimensions using the Manning formula, because this formula has the form very simple but gives a satisfying result. Therefore, this formula can be used widely as a formula for calculating channel capacity.

$$Q = A.V = A \times \frac{1}{n} \times R^{\frac{2}{3}} \times S^{\frac{1}{2}}$$

To be able to use the n-value in Manning, one must meet the requirement first. The limitation for Manning is the total width of the top flood stage should be less than 100 ft or about 30 m. The widest channel is 6 m; meanwhile, the ratio of the side triangle is 1:1, which makes the side length the same as the height = 1.9 m. 6 + (21.9) = 9.8 m is less than 30 m, so it can continue using the Manning formula. The following is an example of the calculation on the first segment of the drainage. All calculation results are shown in Table 7.

- Determining n

The n-value can be obtained from (n = 0.025) [7]

- Determining A

The total area of drainage can be calculated through trapezoid area formula:

$$A = (b + mh) \times h = (4 + 1 \times 1.9) \times 1.9 = 11.21\ m^2$$

- Determining S

$$S = \frac{\Delta h}{L} = \frac{0.188}{640} = 0.00029$$

- Determining R

Radius can be calculated with equation:

$$R = \frac{A}{P} = \frac{A}{b + 2h\sqrt{(1 + m^2)}} = \frac{11.21}{4 + 2 \times 1.9\sqrt{(1 + 1^2)}} = \frac{11.21}{9.374} = 1.1959\ m$$

- Determining V

Manning formula for velocity:

$$V = \frac{1}{n} \times R^{\frac{2}{3}} \times S^{\frac{1}{2}} = \frac{1}{0.025} \times 1.1959^{\frac{2}{3}} \times 0.00029^{\frac{1}{2}} = 0.772\ m/s$$

$$Q = A \times V = 11.21 \times 0.772 = 8.658\ m^3/s$$

Table 7 Capacity of existing drainage

Segments	L (m)	Δh (m)	Sx	n	b (m)	h (m)	m	A (m^2)	P	R	V (m/s)	Q (m^3/s)
1	640	0.188	0.000294	0.025	4	1.9	1	11.21	9.374	1.196	0.772	8.658
2	671	0.199	0.000300	0.025	4	1.9	1	11.21	9.374	1.196	0.776	8.700
3	812	0.253	0.000312	0.025	4	1.9	1	11.21	9.374	1.196	0.795	8.917
4	972	0.243	0.000250	0.025	6	1.9	1	15.01	11.374	1.320	0.761	11.421
5	1034	0.237	0.000229	0.025	6	1.9	1	15.01	11.374	1.320	0.729	10.936
6	986	0.221	0.000224	0.025	6	1.9	1	15.01	11.374	1.320	0.720	10.815

As mentioned before, a primary channel should be able to accommodate the amount of rainfall for at least in 10 years period. Therefore, it should be compared the previous calculation of the flood discharge plan and existing capacity (Table 8).

Table 8 Flood discharge plan compared with existing drainage capacity

Segments	$Q_{existing}$	QT = 10	Status
1	8.658	4.736	Safe
2	8.700	8.432	Safe
3	8.917	12.722	Overflow
4	11.421	14.861	Overflow
5	10.936	17.663	Overflow
6	10.815	19.719	Overflow

From Table 8 above, one can see that there is a problem starting from the third segment; the drainage can no longer hold the amounts of rainfall. Indeed, as seen from Fig. 4 and 5 that the design plan has the safety parameter on both sides of the drainage bank with a 2.4 m dyke. But in the actual case of what has been built, the dyke was never there; this happened possibly because of the settlements around the drainage that are too close.

3.3 Normalization plan

Normalization plan is done by deepening the channel, because widening cannot be done since there are many settlements at the drainage bank. After this normalization, it is hoped that the channel can accommodate the planned flood discharge so flood-prone areas can be overcome. For overflowing channels, the redesign planning is determined by "trial and error" (Table 9). Below is an example of a redesign calculation on the 3rd segment of the drainage system: h will be added to increase the channel capacity (h = 2.5 m), the downstream channel will be (h = 2.7 m).

- Determining A

$$A = (b + mh) \times h = (4 + 1 \times 2.5) \times 2.5 = 16.25\ m^2$$

- Determining S

$S = 0.000312$ see Table 6

- Determining R

$$R = \frac{A}{P} = \frac{A}{b+2h\sqrt{(1+m^2)}} = \frac{16.25}{4+2\times 2.5\sqrt{(1+1^2)}} = \frac{16.25}{11.07} = 1.468\ m$$

- Determining V

$$V = \frac{1}{n} \times R^{\frac{2}{3}} \times S^{\frac{1}{2}} = \frac{1}{0.025} \times 1.468^{\frac{2}{3}} \times 0.00029^{\frac{1}{2}} = 0.912\ m/s$$
$$Q = A \times V = 16.25 \times 0.772 = 14.819\ m^3/s$$

Table 9 Drainage capacity of normalization design

Segments	L (m)	Δh (m)	Sx	n	b (m)	h (m)	m	A (m2)	P	R	V (m/s)	Q (m3/s)
1	640	0.188	0.000294	0.025	4	1.9	1	11.21	9.374	1.196	0.772	8.658
2	671	0.199	0.000297	0.025	4	1.9	1	11.21	9.374	1.196	0.776	8.700
3	812	0.253	0.000312	0.025	4	2.5	1	16.25	11.071	1.468	0.912	14.819
4	972	0.243	0.000250	0.025	6	2.5	1	21.25	13.071	1.626	0.874	18.582
5	1034	0.237	0.000229	0.025	6	2.5	1	21.25	13.071	1.626	0.837	17.792
6	986	0.221	0.000224	0.025	6	2.7	1	23.49	13.637	1.723	0.861	20.214

To see if the normalization of primary drainage capacity can overcome the flood discharge plan, it has to be compared side-by-side with a 10-years period of flood discharge plan (Table 10).

Table 10 Flood discharge plan compared with normalization drainage capacity

Segments	$Q_{re\text{-}design}$	QT = 10	Status
1	8.658	4.736	Safe
2	8.700	8.432	Safe
3	14.819	12.722	Safe
4	18.582	14.861	Safe
5	17.792	17.663	Safe
6	20.214	19.719	Safe

The normalization design plan for the overflowing drainage segments is shown in Fig. 6, Fig. 7 and Fig. 8 as follows.

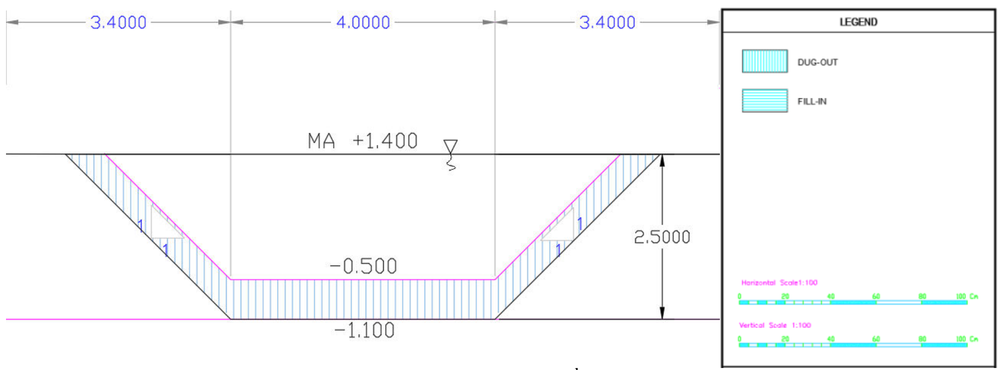

Fig. 6 Normalization plan for 3rd drainage segment

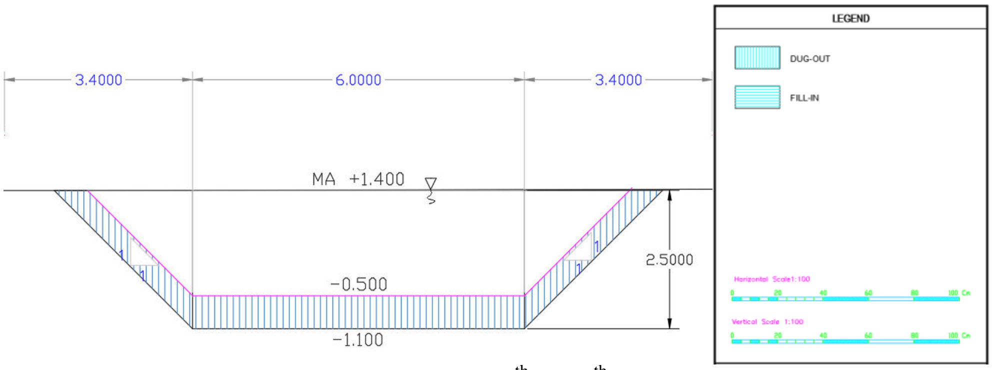

Fig. 7 Normalization plan for 4th and 5th drainage segments

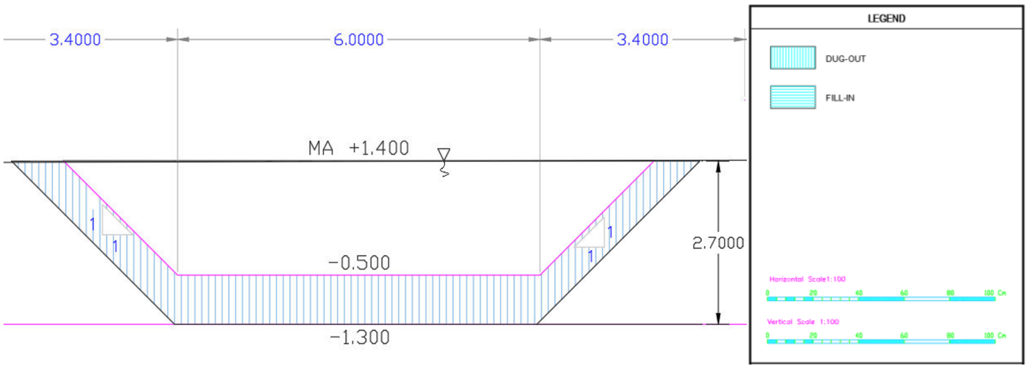

Fig. 8 Normalization plan for 6th drainage segment

4. Conclusions

From the study that has been done, there are some conclusions that can be drawn:

- After analyzing the flood discharge plan in the Merauke city drainage system, known that the flood discharge plan for 10 years is 19.719 m^3/s while the current drainage capacity is only 10.815 m^3/s.
- After the full bank capacity analysis, it is known that the 3rd - 6th segments are overflown.
- The drainage was not built precisely according to the design. In reality, the dykes on the drainage banks have never been made in the first place.
- The growing population of Merauke city is considered to have no impact on land subsidence.
- To overcome the flood-prone areas, carried out a normalization by increasing the depth of the channel on the overflown segments of the drainage system.
- This study's normalization plan is only one of many possibilities to overcome the problem.

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