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Makanuai river flood control study at Jayapura district

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Abstract. The location of this research was carried out in the Makanuai River, Sentani Timur district, Jayapura Regency. Analysis of the planned flood discharge on the Makanuai river using the Rational method for calculating the planned flood discharge. Based on the results of the analysis of the calculation of flood plans with the method that has been done, it can be seen the flood discharge of the Sungai Makanuai plan, the return period Q 2 years = 123,358 m³ / sec, Q 5 years = 155,098 m³ / sec, Q 10 Years = 177,421 m³ / sec, Q 25 Years = 207,294 m³ / sec and Q 50 Years = 230,698 m³ / sec. Based on the analysis of the flow hydraulics on the cross-section of the Makanuai River in the HEC-RAS Program, it was found that the elevation of the floodwater level exceeded the capacity of the river. Therefore, it is necessary to make a Dividing Channel (Corner) accommodate the remaining flow of airflow that cannot be accommodated in the Main Canal so that the capacity of the Makanuai river flow will be reduced when maximum rainfall occurs. The operated Shaving Channel is capable of flowing discharge of 204,259 m³ / sec for a 50 year return period.

1. Introduction

Flooding is a condition where water is not accommodated in the sewer [1]. Flood Control is part of the management of more specific water resources to control flood discharge generally through flood control dams, or improvement of the carrier system (river, drainage) and prevention of potentially damaging things by managing land use and flood areas [2]. High rainfall intensity results in water discharge in the Makanuai River Basin (DAS) experiencing maximum discharge. In early 2019, there was a flood that was greater than the previous flood. The Makanuai River is located right next to the construction of the Istora Papua Bangkit Stadium (PON Venue). Flood control was carried out to reduce standing water so as not to disturb the community and the construction of the Istora Papua Bangkit Stadium (Venue PON) by making a 900-meter long river to drain the remaining runoff that exceeds the capacity of the Makanuai River, Jayapura Regency.

2. Wave theory

2.1. Hydrological Analysis

Hydrological data analysis is intended to determine the hydrological characteristics of the watershed that will be used as a basis for analysis. To determine the flood discharge plan. In determining the rainfall data from the registrar or penakar only obtained rainfall at a certain point (point rainfall). From the results of the calculation of maximum daily rainfall, it is necessary to estimate the maximum amount of rainfall with a certain return period or the possibility of a repeat of the maximum rainfall, for that frequency analysis is carried out. Rainfall intensity is the rainfall that occurs at one unit of time. The intensity of rainfall is calculated on the duration of rain (duration) and its frequency or known as the

curve duration of the frequency of frequency (IDF Curve). The intensity of rainfall is needed to determine the amount of surface runoff. The Mononobe formula is shown in equation 1.

$$I = \frac{R_{24}}{24}\left(\frac{24}{t_c}\right)^{2/3}$$ (1)

Where:
I = rainfall intensity (mm / hour)
TC = rain time or duration (minutes)
R24 = Maximum rainfall in 24 hours (mm)

To calculate the planned peak discharge the Rational Method (RM) is used in which the hydrological data gives a uniform Intensity Duration Frequency (IDF) curve with the peak flow from the average rainfall according to the time of concentration. Peak discharge can be formulated as shown in equation 2.

$$Q = 0.278 \times C \times I \times A$$ (2)

Where :
0.278 = confidence factor (f)
Q = Peak plan discharge
I = intensity (mm / hour)
A = Catchment Area (ha)
C = Runoff coefficient

Analysis of river hydraulics is intended to analyze the profile of floodwater levels in rivers with various times of return from the discharge plan. In the hydraulic analysis will be analyzed how far the influence of flood control that occurs. To support the analysis of river hydraulics, topographic measurements are carried out along the river in question, namely the measurement of the situation, the lengthwise, and the cross-section.

2.2. HEC-RAS
HEC - RAS stands for Hydraulic Engineering Center-River Analysis System. This program was created by the Hydrologic Engineering Center (HEC) which is a division within the Institute for Water Resources, under the US Army Corps of Engineers (USAGE). The HEC-RAS is a steady and unstable one-dimensional flow model [3].

3. Research method
Primary data collection is the measurement data of the Makanuai river obtained from the Papua River Basin. For secondary data is rainfall data obtained from the Climatology and Geophysics Agency (BMKG) of Jayapura Regency and the Makanuai River Basin (DAS) map obtained from the Papua River Basin.

3.1. Hydrological analysis
In the hydrological analysis, the first step that must be done is to process the rainfall data that already exist [4]. After that, determine the statistical parameters (Sd, Cs, Ck, and Cv) to choose the appropriate rainfall frequency distribution method. Rainfall frequency distribution referred to in this case is the normal method, normal log, type III log person, and Gumbel. After obtaining a rainfall distribution method that matches the criteria, the next step is to test the accuracy of the results of the method using the Chi-Square method and look for the hourly rain distribution using the mononobe method. Furthermore, these results are used to find the flood discharge plan with the Rational Method.

3.2. Hydraulics analysis
River cross-section hydraulic analysis is calculated using the HEC-RAS program. Data needed in river cross-section analysis.
1. Cross-section of the river.
2. Manning the cross-section of the river.

3. Maximum existing river debit using the Manning formula.

4. Result and discussion

4.1. Analysis of regional rainfall
From the data obtained the influence of the Sentani rainfall station is 100%, Sentani rainfall station 0%, and east Koya rainfall station 0%. Sentani rainfall data is taken as data used for the calculation of regional rainfall. Daily data collection is obtained from BMKG data. Sentani rainfall data with monthly rainfall data is recapitulated to the maximum daily rainfall, daily data rainfall per month, and daily rainfall data per year. Sentani station rainfall data from Sentani Airport BMKG Station shown in table 1.

Table 1. Sentani station rainfall

No	Years	R (max)
1	2010	83.5
2	2011	139.5
3	2012	89.5
4	2013	126.0
5	2014	108.3
6	2015	85.9
7	2016	75.0
8	2017	65.1
9	2018	81.1
10	2019	66.9

The provisions in the selection of rainfall distribution are based on the criteria listed in the distribution selection parameter table by looking at the calculation results of the statistical parameters that have been made. Based on the parameters of the rain data it can be estimated that the rainfall distribution is appropriate, while the appropriate distribution in this calculation is the Log Pearson Type III method. To strengthen the rainfall distribution estimates chosen, distribution tests were carried out using the Chi-square test and the Smirnov-Kolmogorov test. Calculation of Rainfall Statistics Period 2010 – 2019 shown in table 2.

Table 2. Calculation of Rainfall Statistics Period 2010 - 2019

No	Periode	X (mm)
1	2	89.79
2	5	112.89
3	10	129.14
4	25	150.88
5	50	167.92

In this analysis, the rational method is used to complete the calculation of the flood discharge plan analysis. The recapitulation of planned flood discharge is shown in table 3.

Table 3. Recapitulation of planned flood discharge

Period	I (mm/hours)	C	A	Q(m³/s)
2	25.02	0.62	19.59	84.475
5	31.46	0.62	19.59	106.209

10	35.98	0.62	19.59	121.497
25	42.04	0.62	19.59	141.953
50	46.79	0.62	19.59	157.979
100	55.39	0.62	19.59	187.040

4.2. Analysis of existing cross-sections

After knowing the scope of work, it is necessary to conduct a cross-section analysis existing Siborogonyi river using the Manning formula as shown in equation 3. The calculation of the existing cross-section discharge with the Manning formula is shown in table 4. Elevation of a flood discharge plan for a 2-years return period, 2-years return period, 10-years return period, 25-years return period, 50-years return period shown in table 5 – table 9. Calculation of the dividing channel debit with the manning formula shown in table 10. Sudetan HEC-RAS analysis and sudetan plan shown in figure 1 and figure 2.

$$Q = A \; x \; \frac{1}{n} \; x \; R^{\frac{2}{3}} \; x \; I^{\frac{1}{2}} \tag{3}$$

Table 4. Calculation of the existing cross-section discharge with the Manning formula

STA	V (m/s)	Channel discharge (m³/sec)
0	1.037	18.485
50	0.592	8.661
100	0.638	21.404
150	0.942	31.896
200	0.864	23.407
250	1.062	33.187
300	1.363	39.592
350	1.114	7.525
400	0.827	12.957
450	0.867	14.382
500	3.133	103.335

Table 5. elevation of a flood discharge plan for 2-years return period

River Sta	Q Total (m³/s)	River bed elevation (m)	Water level elevation flood (m)
500	152.59	92.75	101.50
450	152.59	93.47	101.50
400	152.59	93.92	101.49
350	152.59	94.67	101.48
300	152.59	95.08	101.47
250	152.59	95.32	101.46
200	152.59	95.53	101.46
150	152.59	96.32	101.42
100	152.59	96.15	101.43
50	152.59	96.99	101.33
0	152.59	97.54	100.31

Table 6. Elevation of a flood discharge plan for a 5-years return period

River Sta	Q Total (m³/s)	River bed elevation (m)	Water level elevation flood (m)
500	191.85	92.75	101.99
450	191.85	93.47	101.99
400	191.85	93.92	101.97
350	191.85	94.67	101.97
300	191.85	95.08	101.94
250	191.85	95.32	101.94
200	191.85	95.53	101.94
150	191.85	96.32	101.88
100	191.85	96.15	101.90
50	191.85	96.99	101.77
0	191.85	97.54	100.70

Table 7. Elevation of a flood discharge plan for a 10-years return period

River Sta	Q Total (m³/s)	River bed elevation (m)	Water level elevation flood (m)
500	219.46	92.75	102.30
450	219.46	93.47	102.30
400	219.46	93.92	102.29
350	219.46	94.67	102.28
300	219.46	95.08	102.25
250	219.46	95.32	102.24
200	219.46	95.53	102.25
150	219.46	96.32	102.18
100	219.46	96.15	102.20
50	219.46	96.99	102.05
0	219.46	97.54	100.92

Table 8. Elevation of a flood discharge plan for a 25-years return period

River Sta	Q Total (m³/s)	River bed elevation (m)	Water level elevation flood (m)
500	256.41	92.75	256.41
450	256.41	93.47	256.41
400	256.41	93.92	256.41
350	256.41	94.67	256.41
300	256.41	95.08	256.41
250	256.41	95.32	256.41
200	256.41	95.53	256.41
150	256.41	96.32	256.41
100	256.41	96.15	256.41

50	256.41	96.99	256.41
0	256.41	97.54	256.41

Table 9. Elevation of a flood discharge plan for a 50-years return period

River Sta	Q Total (m³/s)	River bed elevation (m)	Water level elevation flood (m)
500	285.36	92.75	102.99
450	285.36	93.47	102.99
400	285.36	93.92	102.97
350	285.36	94.67	102.96
300	285.36	95.08	102.92
250	285.36	95.32	102.91
200	285.36	95.53	102.92
150	285.36	96.32	102.83
100	285.36	96.15	102.86
50	285.36	96.99	102.66
0	285.36	97.54	101.38

Table 10. Calculation of the dividing channel debit with the manning formula

STA	V (m/s)	Cross-section discharge (m³/sec)
0	3.250	204.259
300	3.250	204.259
600	3.250	204.259
900	3.250	204.259

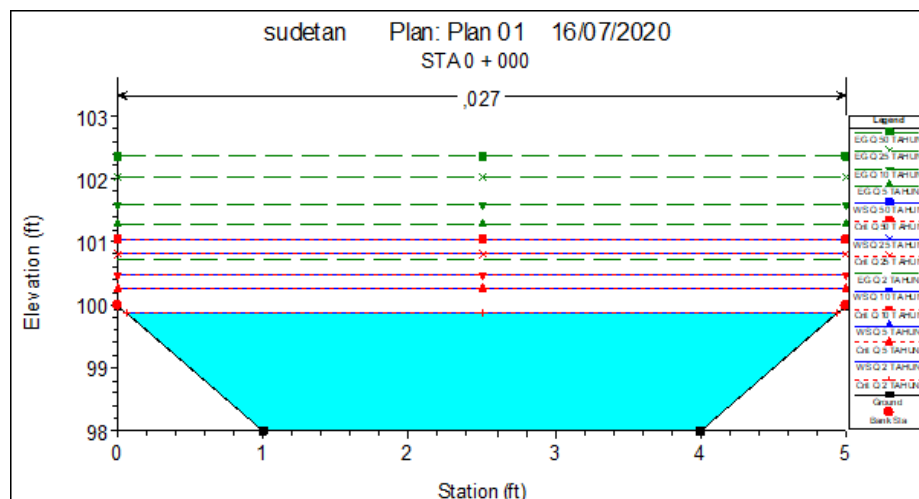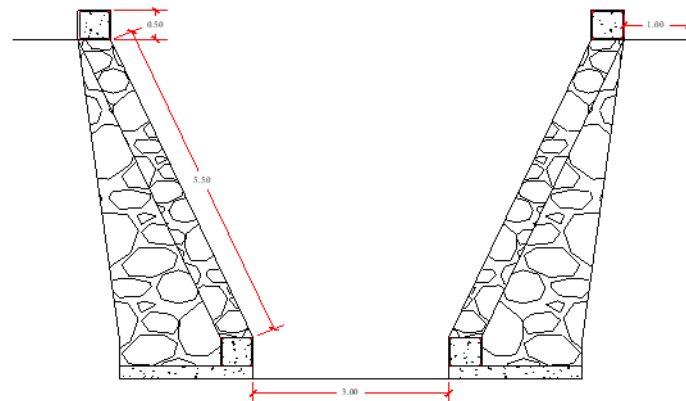**Figure 1.** Sudetan HEC RAS analysis

Figure 2. Sudetan Plan

5. Conclusions

The results of the calculation of the flood discharge plan with the river Rational method Makanuai Q 2 year return period 123,358 m³ / sec, Q 5 year return period 155,098 m³ / sec, Q 10 year return period 177,421 m³ / second, Q 25 year return period = 207,294 m³ / sec . Q 50 Year return Period = 230,698 m³ / sec. Discharge that must be accommodated in the Distribution Channel (Corner) is a return period Q 2 years = 94.737 m³ / sec, Q 5 years = 126,477 m³ / sec, Q 10 Years = 148,800 m³ / sec, Q 25 Years = 178,673 m³ / sec and Q 50 years = 202,077 m³ / sec. Runoff discharge that can be accommodated by sudetan is 204,259 m³ / sec. Dimensions of planned dimensions are Width (b) = 3 meters, Height (h) = 5.50 meters, and guard height (w) = 0.5 meters.

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