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Simulation Model of the Arafuru Retention Pond Flap Gate with Various Discharge Variations

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Abstract

This researchaims to analyze the design flood discharge (Qp) for different return periods based on rainfall intensity and to evaluate flow characteristics using the Froude number (Fr) under various flap gate opening scenarios. Hydrological analysis was conducted using maximum rainfall data from the BMKG Kenten Station for the 2004–2023 period. Frequency analysis employed the Normal, Log-Normal, Log Pearson Type III, and Gumbel probability distributions, with goodness-of-fit evaluated using the Chi-Square and Smirnov-Kolmogorov tests. Design flood discharges were calculated for the 2-year (Q2) and 5-year (Q5) return periods and subsequently applied as input for laboratory hydraulic experiments using an open-channel flume equipped with a 0.6 m × 0.3 m flap gate. Gate opening variations of 25%, 50%, 75%, and 100% were tested. The results indicate that gate opening significantly influences upstream and downstream water levels, flow velocity, and the Froude number. These findings provide a technical basis for determining the optimum flap gate opening to regulate inflow into the Arafuru Retention Basin, thereby improving urban flood mitigation performance in the Buah Sub-watershed, Palembang City.

***Keywords**: urban flooding, retention basin, design flood discharge, Froude number, flap gate, hydraulics.*

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I. INTRODUCTION

Water is a basic human need for survival. Therefore, without it, no one can survive. Water is crucial not only for humans but also for all living things, including animals and plants. With the ever-increasing population, the need for clean water will also increase. Water resource management policies and their impact on water demand are discussed. The results show that effective policies can help improve water use efficiency and reduce waste (Sharma &Anggarwal, 2020).

Flooding is one of the most common hydrological problems occurring in various urban and rural areas of Indonesia. This phenomenon is not only caused by natural factors but is also influenced by uncontrolled changes in land use without considering the sustainability of the watershed from upstream to downstream areas. In flood control, attention should not only be given to surface flow but also to runoff, which is influenced by the intensity and spatial distribution of rainfall throughout the watershed.

The Directorate General of Water Resources has identified several cities in Indonesia with a high level of flood vulnerability, one of which is Palembang City. Based on data from the Palembang City Public Works Agency for Highways and Water Resources Management, several areas within the Buah Watershed (DAS Buah) are included among the priority locations for flood mitigation. Significant changes in land use over the past several years have contributed to an increase in flooding events in the area. In addition, swamp reclamation and the reduction of green open spaces have further increased the risk of inundation, resulting in considerable flood depths and durations.

Several problems have been identified at the Arafuru Retention Pond, including the growth of uncontrolled vegetation and sediment accumulation, which reduce the water storage capacity of the pond. Furthermore, the existing water gates are not functioning properly, resulting in suboptimal flow regulation. The imbalance between the number of inlet and outlet gates also affects the flow patterns entering and leaving the retention pond. Therefore, an assessment of the most effective water-gate opening configuration is required to regulate the inflow discharge.

High annual rainfall generates runoff around the retention pond area. Based on the 2-year and 5-year return-period flood discharges (Q_2 and Q_5), it is predicted that the resulting flood discharge cannot be fully accommodated by the retention pond. This condition provides the basis for this study, which aims to determine an appropriate water-gate opening configuration for regulating the discharge entering the retention pond.

The conveyance capacity of the Air Lakitan River was calculated to be approximately 482.74 m^3/s. The results indicate that the existing river system requires flood-control measures to accommodate such high flow rates. Based on the developed Intensity–Duration–Frequency (IDF) relationships, the study emphasizes the importance of designing drainage systems and flood-control structures according to the local rainfall intensity and duration characteristics to ensure that the planned hydraulic capacity is adequate (Syarifudin, A. et al., 2020).

II. MATERIAL AND METHODS

2.1. Research Location

This research was conducted within the Buah Watershed (DAS Buah), located in Palembang City, South Sumatra, Indonesia. The Buah Watershed extends from the upstream to the downstream areas, covering a total area of approximately 1,223 hectares.

Administratively, the study area encompasses three subdistricts, namely Ilir Timur II Subdistrict, Kalidoni Subdistrict, and Sako Subdistrict. The Buah Watershed is directly adjacent to several surrounding watersheds, namely the Borang Watershed, Juaro Watershed, Bendung Watershed, and Kidul Watershed, as shown in Figures 1 and 2.

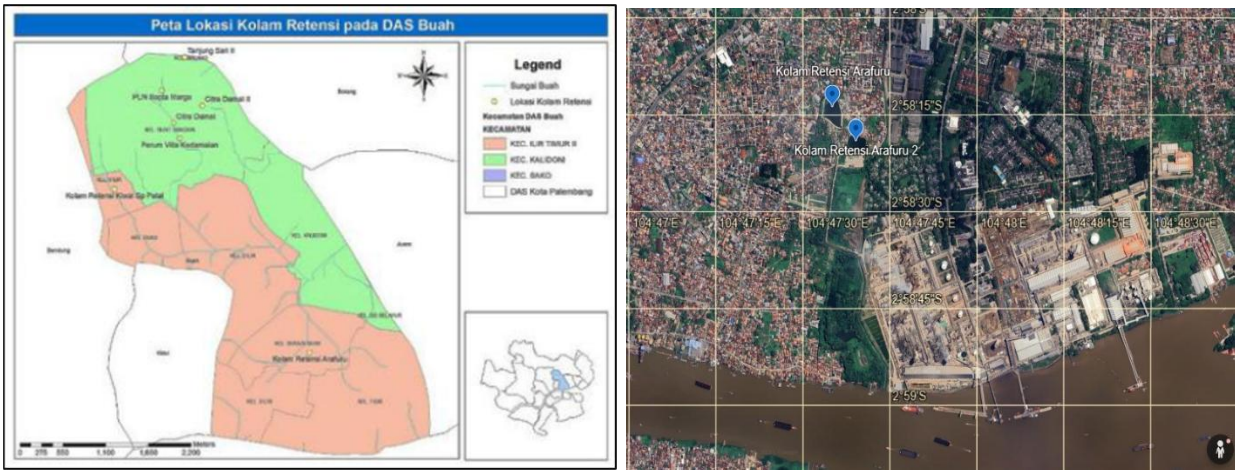

Figure 1: the BuahSub-Watershed and the Arafuru retention pond

2.2 Methods and Data

Currently, there are six existing retention ponds operating along the Buah Sub-Watershed drainage system, namely Kiwal Retention Pond, PLN SaptaMarga Retention Pond, Citra Damai Retention Pond, Citra Damai II Retention Pond, Kedamaian Retention Pond, and Arafuru Retention Pond.

The data collection method used in this study consists of two types of data: primary data and secondary data. Primary data were obtained through a series of direct laboratory experiments using an open-channel hydraulic flume available at the Hydraulics Laboratory, Bina Darma University, Palembang. Meanwhile, secondary data consisted of periodic rainfall records for the period 2004–2023, obtained from the Meteorology, Climatology, and Geophysical Agency (BMKG) of South Sumatra Province.

The research data analysis was conducted using two interconnected methods, namely the analytical method and laboratory experimental method. These two methods are closely related because the flow discharges used in the laboratory experiments were determined based on the results of the hydrological analysis. The design discharges used in the laboratory experiments were the 2-year return-period discharge (Q_2) and 5-year return-period discharge (Q_5). The hydrological analysis consisted of several stages, including rainfall analysis, rainfall frequency analysis, rainfall data distribution testing, rainfall intensity analysis, and determination of the design flood discharge (Q).

The laboratory experiments used a rectangular sluice gate as the test specimen, with a width of 60 cm and a height of 30 cm. The gate was installed vertically across the open channel. The experiments were conducted using various flow discharge rates and gate-opening configurations. Two flow-discharge variations were applied, namely Q_2 and Q_5, while the gate opening consisted of five different opening variations. Thus, the laboratory testing was conducted to evaluate the hydraulic response of the retention pond gate under different combinations of flow discharge and gate-opening conditions.

2.3 Laboratory Test Specimen Scheme

The hydraulic testing in this study was conducted using an open-channel laboratory flume. The test specimen consisted of one sluice gate, which was installed at a distance of 3 m from the upstream water tank within the open channel. The sluice gate was installed vertically across the open channel (*flume*) at a distance of 3 m from the upstream collection tank. The laboratory simulation was conducted using five gate-opening variations and two flow-discharge variations.

The experimental data were subsequently analyzed using analytical, numerical, and computational methods. Several variables were considered as references in the laboratory experiment, consisting of independent variables and dependent variables. The independent variables investigated in this study were the gate opening and inflow discharge (Qin). Meanwhile, the dependent variables consisted of the outflow discharge (Qout), upstream water depth (hu), downstream water depth (hi), and flow velocity. The experimental setup was designed to evaluate the hydraulic characteristics and flow response of the sluice gate under various combinations of inflow discharge and gate-opening conditions.

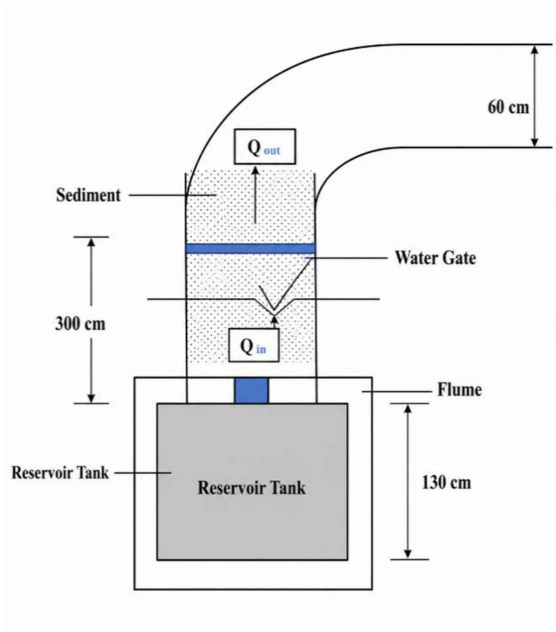

Figure 2: Cross-Sectional Sketch of the Hydraulic Gate Test in an Open Channel

2.4 Flowchart

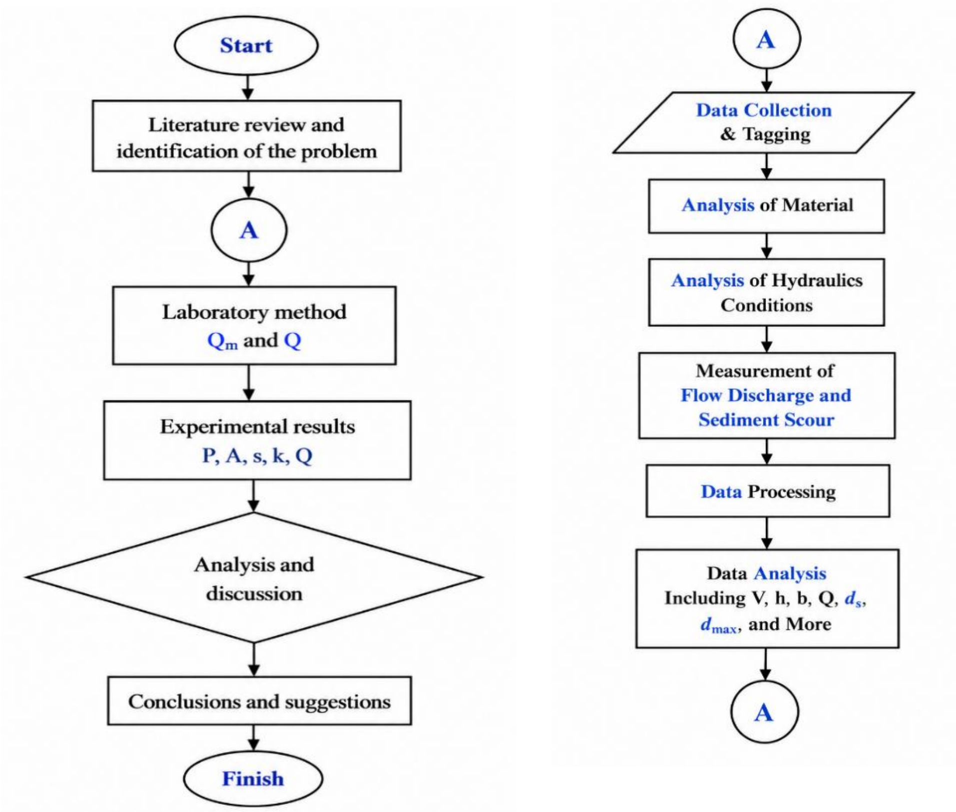

Figure 3: Research Flowchart

2.2. Rainfall Data Analysis

The rainfall data used in this analysis were obtained from the Meteorology, Climatology, and Geophysical Agency (BMKG) Kenten Station, consisting of annual maximum rainfall data over a 15-year observation period (Table 1).

Table 1. Rainfall Data

Year	Station (Xi)
2010	151
2015	138
2013	134
2012	115
2006	114.5
2004	105.5
2016	104
2023	95.4
2005	95
2008	89
2022	86.9
2014	85
2007	85
2009	83.5
2018	82
2021	77.3
2011	75
2020	69.7
2017	55.2

III. RESULTS AND DISCUSSION

3.1 Frequency Analysis

In the frequency analysis, several statistical parameters of the rainfall data were calculated, including the sum, number of data points (n), mean rainfall (R), standard deviation (S), coefficient of variation (Cv), coefficient of kurtosis (Ck), and coefficient of skewness (Cs).

The data used for the statistical analysis consisted of annual maximum rainfall data from 2004 to 2018, as presented in Table 2.

Table 2. Statistical Analysis of Rainfall Data

Year	Station (Xi)	$Xi - \bar{X}$	$(Xi - \bar{X})^2$	$(Xi - \bar{X})^3$	$(Xi - \bar{X})^4$
2010	151	50.220	2522.048	126657.271	6360728.132
2015	138	37.220	1385.328	51561.923	1919134.776
2013	134	33.220	1103.568	36660.542	1217863.213
2012	115	14.220	202.208	2875.403	40888.237
2006	114.5	13.720	188.238	2582.631	35433.695
2004	105.5	4.720	22.278	105.154	496.327
2016	104	3.220	10.368	33.386	107.504
2005	95	−5.780	33.408	−193.101	1116.121
2008	89	−11.780	138.768	−1634.692	19256.669
2014	85	−15.780	249.008	−3929.353	62005.183
2007	85	−15.780	249.008	−3929.353	62005.183
2009	83.5	−17.280	298.598	−5159.780	89161.004
2018	82	−18.780	352.688	−6623.488	124389.107
2011	75	−25.780	664.608	−17133.605	441704.325
2017	55.2	−45.580	2077.536	−94694.109	4316157.493
Total	1511.7	0.000	9497.664	87178.831	14690446.972
Mean	100.78	—	—	—	—
Number of Data	15	—	—	—	—

The statistical parameters of the rainfall data were calculated using Equation (a) as follows:

Number of rainfall data points = 15

Standard deviation (S): $= \frac{1}{n}\sum Xi = 100.78$

Standar Deviasi (S) $= \left[\frac{1}{n-1}\sum_{n-1}^{n}(X - \bar{X})^2\right]^{\frac{1}{2}} = 26.046$

Coefficient of Variation (Cv): $= \frac{S}{\bar{X}} = 0.2584$

Coefficient of Skewness (Cs): $= \frac{n\sum_{n-1}^{n}(X - \bar{X})^3}{(n-1)(n-2)S^3} = 0.4066$

Coefficient of Kurtosis (Ck $= \frac{n^2\sum_{n-1}^{n}(X - \bar{X})^4}{(n-1)(n-2)(n-3)S^4} = 3.288$

Table 3. Statistical Calculation of Logarithmic Rainfall Data

Year	X	Log X	(Log X − Log X̄)²	(Log X − Log X̄)³	(Log X − Log X̄)⁴
2010	151	2.179	0.036	0.006790	0.001286
2015	138	2.140	0.023	0.003393	0.000510
2013	134	2.127	0.019	0.002599	0.000357
2012	115	2.061	0.005	0.000359	0.000026
2006	114.5	2.059	0.005	0.000331	0.000023
2004	105.5	2.023	0.001	0.000038	0.000001
2016	104	2.017	0.001	0.000021	0.000001
2005	95	1.978	0.000	−0.000002	0.000000
2008	89	1.949	0.002	−0.000065	0.000003
2014	85	1.929	0.004	−0.000218	0.000013
2007	85	1.929	0.004	−0.000218	0.000013
2009	83.5	1.922	0.005	−0.000313	0.000021
2018	82	1.914	0.006	−0.000436	0.000033
2011	75	1.875	0.013	−0.001503	0.000172
2017	55.2	1.742	0.061	−0.015193	0.002459
Total	—	29.844	0.113	0.0144	0.004917
Mean	—	1.990	—	—	—

The logarithmic statistical parameters of the rainfall data used for the logarithmic rainfall frequency distribution analysis are calculated as follows:

Log $\bar{X}$ = 1.990

$$S_{\log X} = \left[\frac{1}{n-1}\sum_{n-1}^{n} log(X - \bar{X})^2\right]^{\frac{1}{2}} = 0.090$$

Coefficient of Variation (Cv) $= \dfrac{S_{\text{Log X}}}{\text{Log }\bar{\text{X}}} = 0.133$

Coefficient of Skewness (Cs) $= \dfrac{n\sum_{n-1}^{n} log(X - \bar{X})^3}{(n-1)(n-2){S_{\text{Log X}}}^3} = 0.14$

Coefficient of Kurtosis (Ck) $= \dfrac{n^2\sum_{n-1}^{n}(X - \bar{X})^4}{(n-1)(n-2)(n-3){S_{\text{Log X}}}^4} = 0.090$

3.2 Rainfall Distribution

The statistical parameter values were obtained from the calculations presented above. The next step was to determine the appropriate probability distribution to be used in the frequency analysis. Several probability distribution methods were considered, as follows: Normal (mm, Log-Normal (mm), Log-Pearson Type III (mm), Gumbel (mm). From the frequency analysis using the four probability distribution methods described above, it was found that each distribution produced different design rainfall values for each return period. Therefore, a summary of the design rainfall calculations for each probability distribution is presented in Table 8.

Table 4. Summary of Design Rainfall Distributions

Return Period (Years)	Normal (mm)	Log-Normal (mm)	Log-Pearson Type III (mm)	Gumbel (mm)
5	102.343	139.711	110.739	102.580
10	103.161	153.019	127.801	103.949
25	103.943	166.901	154.483	105.677
50	104.594	179.428	178.283	106.960
100	105.115	190.123	205.751	108.233

3.3 Rainfall Intensity Analysis

Rainfall intensity is related to the duration and frequency of rainfall and is generally presented in the form of an Intensity–Duration–Frequency (IDF) curve. The IDF curve can be used to determine rainfall intensity for various return periods, such as 2, 5, 10, 25, 50, and 100 years. The following presents the calculation of rainfall intensity for a 5-year return period.

Table 5. Maximum Rainfall from the Log-Pearson Type III Distribution

Return Period (years)	R_{24} (mm)
5	110.739
10	127.801
25	154.483
50	178.283
100	205.751

3.4 Time of Concentration (Tc)

The time of concentration (Tc) is defined as the time required for a water particle to travel from the hydraulically most distant point within a watershed to the point of interest, which is generally the watershed outlet (Suripin, 2004; Triatmodjo, 2008; Syarifudin, 2018).

The lag time (basin lag) is defined as the time difference between the centroid of effective rainfall and the peak of the unit hydrograph.

Tc = 0.01947 L77* $S^{-0,385}$
= 0.01947*1.277*$0.026^{-0.385}$
= 0.01947*5.39*4.076
= 0.4277 hour

The lag time can also be calculated using the following equation :

tL = 0.6 tc = 0.6 * 0,4277 = 0.2566 jam

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

$$= \left(\frac{110{,}739}{24}\right)\left(\frac{24}{0{,}2566}\right)^{\frac{2}{3}}$$

$$= 96.52 \text{ mm/hour}$$

The rainfall intensity calculations were performed for return periods of 5, 10, 25, 50, and 100 years, with rainfall durations evaluated at 10-minute intervals. The resulting values are used to develop the Intensity–Duration–Frequency (IDF) curve As shown in Figure 5.

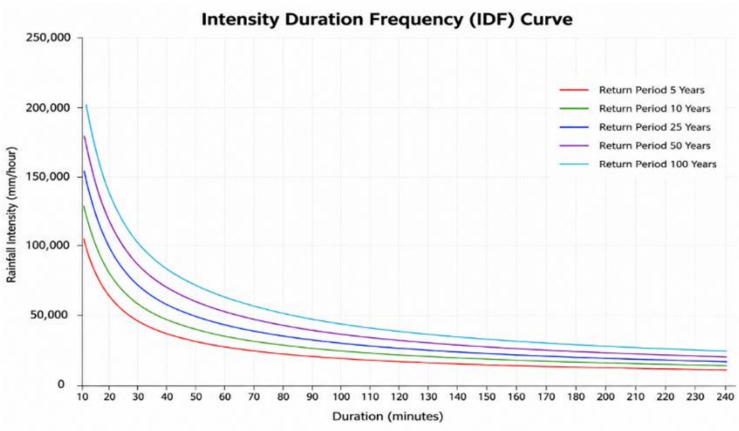

Figure 5: Intensity–Duration–Frequency (IDF) Curve

3.5 Runoff Coefficient Analysis

Runoff analysis is conducted to determine the surface runoff discharge occurring in the Buah Sub-Watershed.

The surface runoff discharge is calculated using the Rational Method. The runoff coefficient (C) is determined based on the land-use characteristics of the Buah Sub-Watershed.

Table 6. Runoff Coefficients (C) of the Buah Sub-Watershed

No.	Land Use	Area (ha)	Percentage (%)	Runoff Coefficient (C)	Weighted C
1	Fishpond	0.018	0.0000	77	1.386
2	Trees	0.927	0.2559	87	76.941
3	Shrubland	0.963	0.1063	78	75.114
4	Swamp	0.180	0.0199	77	13.860
5	Agricultural Land	2.593	0.2863	01	235.963
6	Settlement	12.080	56.6855	84	1014.720
7	Asphalt Road	1.353	7.4694	98	132.594
Total		18.114	64.832		1550.58

Coefficient of the Buah Sub-Watershed is obtained as C $=\frac{\sum_{i=1}^{n} Ci.Ai}{\sum_{i=1}^{n} Ai}$ = 85,60

Given the runoff coefficient, (C) = 85.60, The area of the Buah Watershed is 10.97 km^2 therefore:
Q = 0.2778.C.I.A
= 0.2778 * 85.60 *96.52 *19.97
= 5.784910239 m^3/Sec

For return periods of 5, 10, 25, 50, and 100 years, the results of the design flood discharge analysis are presented in Table 7.

Table 7. Results of Design Flood Discharge Analysis

Return Period (years)	(mm)	C	(mm/hour)	(km²)	(m³/sec)	(m³/sec)
5	110.74	85.6	96.52	10.97	25.17	0.0025
10	127.80	85.6	111.39	10.97	29.05	0.0029
25	154.48	85.6	134.65	10.97	35.12	0.0035
50	178.28	85.6	155.40	10.97	40.53	0.0040
100	205.75	85.6	179.34	10.97	46.78	0.0046

For the design flood discharge calculation, the 10-year return period was selected, with a design flood discharge of: Qp = 29,05 m^3/secorQm = 0,0029 ml/sec. The results are presented in Table 8.

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Table 8. Discharge with Various Gate Openings

Simulation	Gate Opening (%)	(m³/sec)	(m³/sec)
1	25	7.2647	0.0073
2	50	14.5293	0.0145
3	75	21.7940	0.0218
4	100	29.0586	0.0291

3.6 Simulation with Q_{model}= 0,0073 m³/sec (25% Gate Opening)

With a wetted cross-sectional area of 0.0096 m² and an average discharge of 2,1281 × 10^{-3}m³/sec, the flow velocity is obtained as: v=0,029 m/sec, This flow velocity is relatively low, indicating that the kinetic energy of the flow is relatively small. The flow velocities corresponding to the various model discharges are presented in Table 9.

Table 9. Flow Velocity for Various Discharges

Q (m³/sec)	A (m²)	V (m/sec)
0.007264659	0.0096	0.75673530
0.014529318	0.0096	1.51347060
0.021793977	0.0096	2.27020591
0.029058636	0.0096	3.02694121

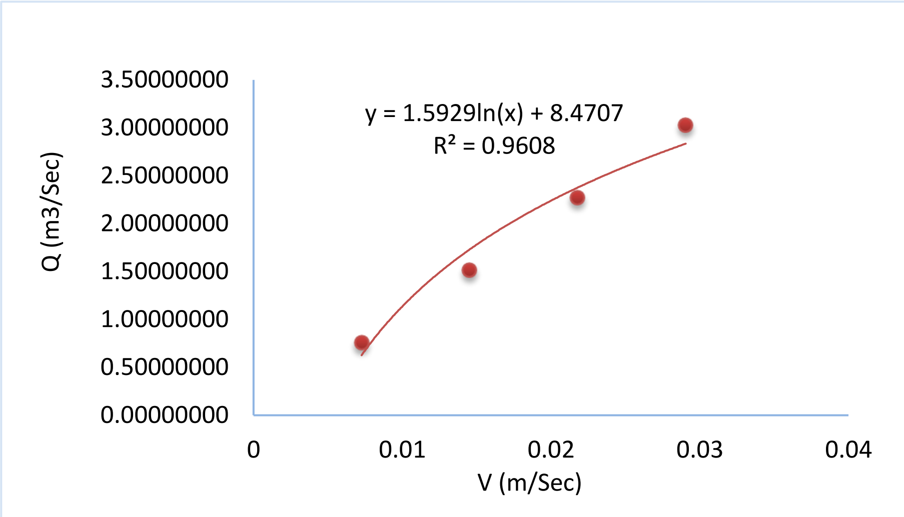

Figure 6: Relationship between Discharge (m³/sec) and Flow Velocity (m/sec)

IV. CONCLUSION

Based on the data analysis and discussion, the conclusions of this study are as follows:

1. The calculated flow discharges for various return periods, based on the rainfall intensity values, are Q5 = 0,0025 m3/s; Q10 = 0,0029 m3/s; Q25 = 0,0035 m3/s; Q50 = 0,0040 m3/s and Q100 = 0.0046 m3/s.
2. The flow characteristics based on the Froude number (Fr) for various flap gate openings are all classified as subcritical flow (Fr < 1). The Froude numbers are 7.4×10^{-6} for a 25% gate opening, 3.7×10^{-5} for 50%, for 5.6×10^{-5} for 75%, and 7.4×10^{-5} for 100%.
3. These results indicate that the flow remains in the subcritical or tranquil regime, characterized by relatively low flow velocity and low scour potential. Therefore, under the tested conditions, the flap gate is considered capable of controlling the flow and reducing the potential for overtopping. Nevertheless, the gate performance should also be evaluated against the design discharge and the actual downstream water-level conditions.

REFERENCES

[1]. Achmad Syarifudin. (2018). Applied Hydrology. Andi Publisher, Yogyakarta, p. 45–48.
[2]. Achmad Syarifudin. (2023). Environmentally Friendly Urban Drainage Systems. Bening Meria Publishing, pp. 49–54.
[3]. Achmad Syarifudin. (2022). Hydraulics and Dimensional Analysis. Bina Darma Press.
[4]. Pratiwi, A. A., & Syarifudin, A. (2023). Analysis of the Arafuru Retention Pond Storage Capacity as a Flood Control Measure in the Buah Sub-Watershed. Bina Darma Conference on Engineering Science.
[5]. Ikhsan, C. (2017). The Effect of Flow Discharge Variations on an Open Channel with Uniform Flow. Media Teknik Sipil.
[6]. Hoirisky, C., Rahmadi, & Harahap, T. (2018). The Effect of Changes in Land-Use Patterns on Flooding in the Buah Watershed, Palembang City. Proceedings of the 2018 National Seminar on World Water Day, Palembang, March 20, 2018. e-ISSN: 2621-7449.
[7]. Alia, F., Iryani, S. Y., & Ramadhanti, N. (2020). Analysis of Retention Pond Capacity for Flood Control in the Buah Watershed, Palembang City. CANTILEVER, 9(2), October 2020. ISSN 1907-4247 (Print); ISSN 2477-4863.
[8]. Suryadinata, I. G. P., Norken, I. N., & Dharma, I. G. B. S. (2013). Evaluation of the Retention Basin Performance Plan as an Effort to Control Flooding in Tukad Mati, Denpasar City. Jurnal Spektran, 1(1), January 2013.
[9]. Nofrizal. (2017). Flood Control Analysis Using a Retention Pond Model in the Batang Kuranji Watershed Using the MIKE FLOOD Program in Padang City. Jurnal MENARA Ilmu, XI(2), No. 77, October 2017.
[10]. Okubo, K., Muramoto, Y., & Morikawa, H. (1994). Experimental Study on Sedimentation over the Floodplain Due to River Embankment Failure. Bulletin of the Disaster Prevention Research Institute, Kyoto University, 44(2), 69–92.
[11]. Siregar, S., Sitompul, H., Wijaya, K., Solahuddin, A. A., & Nurmaidah. (2023). Design of a Retarding Basin as an Effort to Reduce Flooding.
[12]. Syarifudin, A., & Destania, H. R. (2020). IDF Curve Patterns for Flood Control of the Air Lakitan River, Musi Rawas Regency. IOP Conference Series: Earth and Environmental Science, 448. Proceedings of the 1st International Conference on Environment, Sustainability Issues and Community Development, October 23–24, 2019, Central Java Province, Indonesia.