Journal of Social Research

P-ISSN:2827-9832
E-ISSN: 2828-335x
Vol. 5, No. 7, June 2026
http://ijsr.internationaljournallabs.com/index.php/ijsr

Analysis of a Detention Basin as a Flood Control Measure In Kedawung Village, Tanjung Subdistrict, Brebes Regency

Nanda Salsabila, Nurdiyanto
Universitas Swadaya Gunung Jati, Indonesia
Email: nandasalsabila7788@gmail.com

Keywords:
detention basin, flood control, flood routing, flood discharge reduction, sluice gate opening variation

ABSTRACT

Flooding is a common problem in local communities due to heavy rainfall and limited drainage capacity. A detention basin is a flood control infrastructure designed to temporarily store surface runoff before releasing it downstream. This study analyzes the performance of a detention basin as a flood control measure using a flood routing approach, with emphasis on sluice gate opening variation and its effect on outflow behavior. The research employs quantitative hydrological and hydraulic analysis under three operational conditions: fully closed, half-open, and fully open sluice gate. Hydrological analysis includes the determination of design rainfall using a selected probability distribution with goodness of fit testing, followed by calculations of time of concentration, rainfall intensity, and design flood discharge. The resulting discharge is used as inflow in the flood routing simulation. Hydraulic analysis covers outflow computation through the sluice gate and evaluation of storage changes and water surface elevation within the basin. The relationship between inflow and outflow is examined to understand system response under different gate openings over time. The results show that for a 25-year return period, the design flood discharge is 4,060 m^3/s with a time of concentration of 44.861 minutes. The flood discharge reduction under half-open sluice gate conditions reaches 64.4%, indicating a significant influence of gate operation on outflow regulation. In conclusion, detention basins play an important role in flood control through peak attenuation and flow delay, while sluice gate opening variation strongly affects flood routing performance and outflow characteristics.

INTRODUCTION

According to data from the National Agency for Disaster Countermeasure (BNPB), the number of floods that occurred in Indonesia between 2020 and 2024 reached 7,518 incidents. Flooding is a natural disaster caused by high water levels from rivers, the sea, or heavy rainfall, resulting in land being inundated with water (Sunarya & Sutoyo, 2023); however, it can become a disaster when it impacts human settlements and causes losses, whether in the form of loss of life or property damage (Setiawan et al., 2020). The phenomenon of flooding can be observed in the Babakan River, one of the rivers in Brebes Regency that frequently experiences a significant increase in flow during the rainy season. In some instances, this river has even overflowed and flooded the surrounding settlements. This indicates that the Babakan River does not always have the capacity to accommodate the flow during heavy rains. Therefore, river water volume control is required to protect the downstream section from the accumulation of runoff coming from the upstream. The area selected for flood control efforts is also an area that requires management to address the potential for local flooding in that region. One strategic location for such control efforts is in Kedawung Village.

The village of Kedawung, located in Tanjung Subdistrict, Brebes Regency, Central Java, is one of the areas prone to waterlogging and flooding due to the suboptimal drainage system in the surrounding area. Drainage channels that do not function optimally result in limited water flow capacity, which in turn triggers waterlogging and flooding (Nursella & Ariyanto, 2025). The drainage system in this area is not directly connected to the Babakan River, so the flow must pass through small channels with limited capacity. Channel narrowing and blockages can reduce flow capacity, thereby increasing the risk of local waterlogging and flooding during heavy rains.

Flood disasters can disrupt community activities, such as hindering various operations and potentially damaging infrastructure like roads and buildings, which ultimately result in physical and social losses for the community. Meanwhile, from an economic perspective, floods can cause losses in the agricultural and fisheries sectors, damage infrastructure, and lead to business losses (Arashi et al., 2024). Therefore, a flood control system capable of regulating flow in a planned manner is required. One piece of infrastructure that plays a crucial role in such a system is a detention basin. Detention basin serve as temporary reservoirs for rainwater, which is then gradually released into drainage channels (Hasan & Widyanto, 2025).

A number of studies have shown that detention basins can help reduce flooding. Naufal Yandri (2021) demonstrated that detention basins can help reduce water accumulation, although their performance may be affected by tidal conditions. Hasan & Widyanto (2025) also confirmed that detention basins are effective at controlling surface runoff when integrated with a pumping system. Meanwhile, Wigati et al. (2021) designed a retention and detention based drainage system that has been proven to increase flow capacity in urban areas. Another study by Amalia et al. (2023) shows that planning detention basins with adequate storage capacity can accommodate design flood volumes through a hydraulic simulation approach. Furthermore, Marsidi (2024) demonstrated in his study that detention basins are effective in reducing peak runoff discharge, although they are not yet optimal in reducing inundation area and water level at some locations.

However, the performance of detention basins is influenced by the characteristics of the catchment area, storage capacity, and the operating patterns of the sluice gate. Previous studies have generally been limited to specific conditions and locations and have not extensively examined the effect of variations in sluice gate openings on the outflow from detention basins. Therefore, this study aims to analyze the performance of the detention basin in Kedawung Village, Tanjung Subdistrict, Brebes Regency through flood routing simulations, taking into account variations in sluice gate openings in determining outflow, so as to provide more optimal recommendations for flood control efforts in the area.

METHOD

This study is a case study employing a hydrological and hydraulic analysis approach that focuses on the analysis of detention basins as a flood control measure. This study uses a quantitative research method. A quantitative method is an approach that uses statistical tools to process data, so that the information obtained and the results of the analysis are presented in numerical form (Sahir, 2021).

Research Location

Figure 1. Research Location

The research location is at the detention basin in Kedawung Village, Tanjung Subdistrict, Brebes Regency, Central Java Province, with geographical coordinates of 6°54'55.51" S and 108°53'29.92" E.

Method for Filling in Missing Rainfall Data

Method for filling in missing rainfall data is used when data from a particular rain gauge station is incomplete due to technical or non-technical errors. This process is carried out using the normal ratio method, which utilizes data from the nearest rain gauge station.

$$P_x = \frac{1}{n}\left(P_a \frac{A_{nx}}{A_{na}} + P_b \frac{A_{nx}}{A_{nb}} + \cdots P_n \frac{A_{nx}}{A_{nn}}\right)$$

Notes:

P_x = Rainfall at stations with incomplete data (mm)
$P_{a,b}$ = Rainfall at stations a and b (mm)
n = Number of stations around x used to estimate data for x
A_{nx} = Annual rainfall at the station with incomplete data (mm)
$A_{na,b}$ = Annual rainfall at stations a and b (mm)

Analysis of Average Rainfall

Rainfall is the amount of rainwater that falls onto the Earth's surface, assuming that the surface is flat, impermeable, free from evaporation, and evenly distributed, and is expressed as water depth (Soewarno, 2015). In hydrological analysis, rainfall is used to determine the potential for surface runoff and serves as the basis for calculating the average rainfall of a region. According to Suripin (2004), the Thiessen Polygon Method is a method for calculating average rainfall by assigning weights to the areas of influence of rain gauges to account for unevenness.

$$P = \frac{P_1A_1 + P_2A_2 + P_3A_3 \ldots + P_nA_n}{A_1 + A_2 + A_3 \ldots + A_n}$$

Notes:

P = Average rainfall
P_1, P_2, P_3, P_n = Rainfall recorded at rain gauges 1, 2, 3, ..., n
A_1, A_2, A_3, A_n = Area of polygons 1, 2, 3, ..., n

Rainfall Frequency Analysis

Frequency analysis is a statistical approach used to determine the magnitude of design rainfall or design flood with a specific return period (Pudyastuti & Musthofa, 2020). According to (Widyawati et al., 2020), distribution analysis plays a crucial role in determining the type of distribution that best fits the rainfall data, thereby minimizing calculation errors. The selection of the frequency analysis method is performed by calculating the average rainfall, standard deviation, asymmetry coefficient, kurtosis coefficient, and coefficient of variation. The results of these parameters then serve as the basis for determining the method that meets the criteria in the following table.

Table 1. Statistical parameters for determining the type of distribution

No.	Distribution	Requirements
1.	Normal	$(\overline{x}\pm s) = 68{,}27\%$ $(\overline{x}\pm 2s) = 95{,}44\%$ $C_s \approx 0$ $C_k \approx 3$
2.	Log Normal	$C_s = C_v^3 + 3C_v$ $C_k = C_v^8 + 6C_v^6 + 15C_v^4 + 16C_v^2 + 3$
3.	Gumbel	$C_s = 1{,}14$ $C_k = 5{,}4$
4.	Log Pearson III	Appart from the above

Source: Triatmodjo, 2008

The following are the formulas for calculating statistical parameters:

1. Average Rainfall ($\overline{x}$)

$$\overline{x} = \frac{\sum_{i=1}^{n} xi}{n}$$

2. Standard Deviation (S)

$$S = \sqrt{\frac{\sum_{i=1}^{n}(xi - \overline{x})}{(n-1)}}$$

3. Skewness Coefficient (C_s)

$$C_s = \frac{n^2 \sum_{i=1}^{n}(xi - \overline{x})^3}{(n-1)(n-2)s^3}$$

4. Coefficient of Kurtosis (C_k)

$$C_k = \frac{n^2 \sum_{i=1}^{n}(xi - \overline{x})^4}{(n-1)(n-2)(n-3)s^4}$$

5. Coefficient of Variation (C_v)

$$C_v = \frac{S}{\overline{x}}$$

Notes:

$\overline{x}$ = Annual average Rainfall (mm)
n = Number of data
xi = Annual maximum Rainfall (mm)
S = Standard deviation
C_s = Skewness Coefficient
C_k = Coefficient of Kurtosis
C_v = Coefficient of Variation

Analysis of Design Rainfall

The calculation of the design rainfall distribution analysis uses the Log Pearson III distribution formula, as follows:

$$LogX_T = Log\bar{X} + k \; x \; S \; LogX$$

$$Log\bar{X} = \frac{\sum_{i=1}^{n} Logx_i}{n}$$

$$S \; LogX = \sqrt{\frac{\sum_{i=1}^{n}(Logx_i - \; Log\bar{X})^2}{n-1}}$$

$$C_s = \frac{n \; \sum_{i=1}^{n}(Logx_i - \; Log\bar{X})^3}{(n-1)(n-2)(S \log X)^3}$$

Notes:

$LogX_T$ = Logarithmic value of the design rainfall for a return period of T
$Log\bar{X}$ = Logarithmic value of the average design rainfall
k = Standard variable; its value depends on the slope coefficient (C_s)
$S \; LogX$ = Standard deviation of LogX
$Logx_i$ = Value of the i-th rainfall data point
C_s = Skewness coefficient

Distribution Fit Test

The Smirnov-Kolmogorov test is a statistical method used to test whether a data distribution fits a specified distribution. This test is used to determine whether the distribution pattern of the observed data aligns with the distribution type used in the analysis. The test is conducted by comparing the value of the maximum probability difference to the critical value at the 5% significance level.

Concentration Time

Concentration time is the time it takes for rainwater to flow from the farthest point to the drain. Concentration time (t_c) is calculated using the Kirpich formula.

$$t_c = \; 0{,}01947 \; x \; L^{0{,}77} x \; S^{-0{,}385}$$

Notes:

t_c = Concentration time (minutes)
L = Maximum length of the water flow path (m)
S = Average slope of the water flow path

Rainfall Intensity

Rainfall intensity is the amount of rainfall that falls on a given area within a specific time period. This measure is one of the key parameters in hydrology because it indicates how quickly rain falls to the ground. Rainfall intensity (I) is calculated using the Mononobe formula.

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

Notes:

I = Rainfall intensity(mm/h)
R_{24} = Daily rainfall (mm)
t_c = Concentration time (hour)

Design Flood Discharge Analysis

The design flood discharge analysis is conducted by considering the hydrological characteristics of the catchment area (CA) as the basis for determining the amount of surface runoff flowing into the detention basin. A watershed area is a part of a river basin area composed of several sub-river basins, each of which is divided into watersheds as the smallest units in the water drainage system (Gultom et al., 2022). Additionally, land use conditions are a critical factor in calculating design flood discharge. Land use is one of the human induced causes of flooding. Changes in land use resulting from population growth and development cause the ground surface to respond differently to rainfall (Sutiyono, 2021).

Analysis of design flood discharge using the rational formula is conducted to estimate the magnitude of the peak discharge that occurs. According to Suripin (2004), the rational method is a simple and easy to apply approach; however, this method is only applicable to watershed areas with an area of less than 300 ha.

$$Q = 0{,}278\ x\ C\ x\ I\ x\ A$$

Notes:

Q = Peak surface runoff discharge (m^3/s)

C = Runoff coefficient

I = Rainfall intensity (mm/h)

A = Catchment area (km^2)

Detention Basin Storage Capacity

The storage capacity of a detention basin is the volume of water that can be temporarily stored before being discharged through the basin's outlet. In this study, the storage capacity was determined based on design data from the project, which was digitally modeled. The storage volume was calculated to establish the relationship between water level elevation and basin volume, which will later be used as a parameter in flood routing.

Sluice Gate in Outlet Systems

A sluice gate is a mechanical structure installed at the outlet of a detention basin or water channel that serves to regulate the outflow in a controlled manner. This gate can be raised or lowered to adjust the outflow rate as needed, thereby controlling the water level in the basin.

$$Q = C_d\ x\ b\ x\ a\ \sqrt{2gh}$$

Notes:

Q = Outflow (m^3/s)

C_d = Flow coefficient

b = Slide gate width (m)

a = Gate opening (m)

g = Acceleration due to gravity (9.81 m/s^2)

h = Water depth upstream of the gate (m)

The Relationship Between Inlet, Reservoir, and Outlet

The magnitude of the inflow and outflow determines how the volume of water in the basin changes over time and affects changes in the reservoir's volume. Changes in the basin's volume can be expressed using the law of continuity as follows.

$$\frac{dS}{dt} = Q_{in} - Q_{out}$$

Notes:

$\frac{dS}{dt}$ = Change in basin volume (m^3/s)
Q_{in} = Inflow (m^3/s)
Q_{out} = Outflow (m^3/s)

RESULTS AND DISCUSSION

Rainfall Analysis of the Study Area

Based on rainfall data from three rain gauges (Cisadap, Nambo, and Ketanggungan) covering the period from 1998 to 2017, there were missing data at the Ketanggungan rain gauge in 2010, 2011, and 2017. Using the normal ratio method, the rainfall values for the Ketanggungan Rain Gauge Station were determined to be 93,54 mm in 2010; 109,27 mm in 2011; and 165,32 mm in 2017.

Table 2. Areal Rainfall (Thiessen Method, Data Completed Using Normal Ratio)

No.	Year	Nambo Rainfall Station (mm)	Cisadap Rainfall Station (mm)	Ketanggungan Rainfall Station (mm)	Maximum Average Rainfall (mm)
1	1998	104	118	140	140.00
2	1999	135	104	180	180.00
3	2000	110	110	98	98.00
4	2001	155	121	94	94.00
5	2002	92	76	110	110.00
6	2003	105	74	147	147.00
7	2004	105	72	90	90.00
8	2005	130	75	160	160.00
9	2006	101	73	116	116.00
10	2007	102	70	120	120.00
11	2008	90	70	180	180.00
12	2009	81	65	78	78.00
13	2010	85	95	**93.54**	93.54
14	2011	142	73	**109.27**	109.27
15	2012	108	106	114	114.00
16	2013	130	105	115	115.00
17	2014	109	118	210	210.00
18	2015	118	117	123	123.00
19	2016	105	195	125	125.00
20	2017	149	169	**165.32**	165.32

Average rainfall analysis was conducted using the Thiessen Polygon Method processed with ArcMap software (ArcGIS). Based on the spatial delineation results, it was found that the entire 0,272 km² watershed area is entirely within the Ketanggungan Rain Gauge Station's catchment. Since the Ketanggungan Rain Gauge Station is located at the center of the study area, this results in the Ketanggungan Rain Gauge Station having a 100% influence, while the Cisadap and Nambo Rain Gauge Stations do not contribute any weight.

Statistical parameters serve as indicators in selecting the most representative distribution type for the design rainfall data. The use of each distribution type such as the Normal, Log Normal, Gumbel, and Log Pearson III distributions has criteria that must be met.

Table 3. Results of Statistical Distribution Tests

No.	Distribution	Requirements	Result	Description
1	Normal	68,27%	70%	Ineligible
		95,44%	95%	
		0	0.79	
		3	3.33	
2	Log Normal	0.164	0.31	Ineligible
		3.048	2.83	
3	Gumbel	1.14	0.79	Ineligible
		5.4	3.33	
4	Log Pearson III	Cs ≠ 0	0.31	Eligible

Based on the results in the table above, it is evident that the most appropriate distribution is the Log Pearson III.

To verify the fit of this distribution to the field data, a test was conducted using the Smirnov-Kolmogorov method. Based on the goodness of fit test results, a maximum D value of 0,09 was obtained; meanwhile, the critical D value from the Smirnov-Kolmogorov test table with a 5% confidence level and a sample size of 20 is 0,29. Thus, the results of the goodness of fit test are deemed acceptable.

The calculation of design rainfall in this study was performed using the Log Pearson Type III distribution. This method is used to analyze rainfall data based on specific return periods. The results of the Log Pearson Type III distribution were then used to determine design rainfall for return periods of 2, 5, 10, and 25 years.

Table 4. Design Rainfall Distribution Log Pearson III

Return Period, Tr (Year)	Pr (%)	K	K.SdLogR	Log R Design	R Design (mm)
2	50	-0.052	-0.006	2.088	122.456
5	20	0.823	0.094	2.188	154.272
10	10	1.310	0.150	2.244	175.451
25	4	1.853	0.212	2.306	202.484

Analysis of Rainfall Duration and Intensity

The rainfall duration was calculated using the Kirpich method, with a flow path length of 1.406 m and a channel slope of 0,0037%. From these parameters, a value of t_c of 44,861 minutes was obtained. This value will be used further to determine rainfall intensity for the planned return period.

Rainfall intensity analysis was performed using the Mononobe Method. This analysis focused on a 25-year return period as the primary reference for calculating the design flood discharge. This focus was adopted in accordance with the project's technical documents. Calculations for other return periods 2, 5, and 10 years were performed as supporting data to analyze hydrological trends and to test the system's capacity under various rainfall load scenarios. The following is the analysis of the planned rainfall intensity using the Mononobe Method

Table 5. Intensity of Design Rainfall

t (minutes)	Design Rainfall (mm/24 hour)			
	2 Year	5 Year	10 Year	25 Year
	122.456	154.272	175.451	202.484
	(mm/hour)	(mm/hour)	(mm/hour)	(mm/hour)
5	222.517	280.330	318.816	367.937
10	140.177	176.597	200.842	231.786
15	106.975	134.769	153.271	176.886
20	88.306	111.249	126.522	146.016
25	76.100	95.872	109.034	125.833
30	67.390	84.899	96.555	111.431
35	60.808	76.607	87.125	100.548
40	55.629	70.083	79.704	91.984
44.861	51.534	64.924	73.837	85.213
45	51.428	64.790	73.685	85.038
50	47.940	60.395	68.687	79.270
55	44.988	56.677	64.458	74.390
60	42.453	53.483	60.826	70.197
65	40.247	50.704	57.665	66.549
70	38.307	48.260	54.885	63.341
75	36.585	46.090	52.418	60.494
80	35.044	44.149	50.210	57.946
85	33.656	42.400	48.222	55.651
90	32.398	40.815	46.419	53.570

The results of the rainfall intensity calculations are then presented in the form of an IDF (Intensity Duration Frequency) graph as follows.

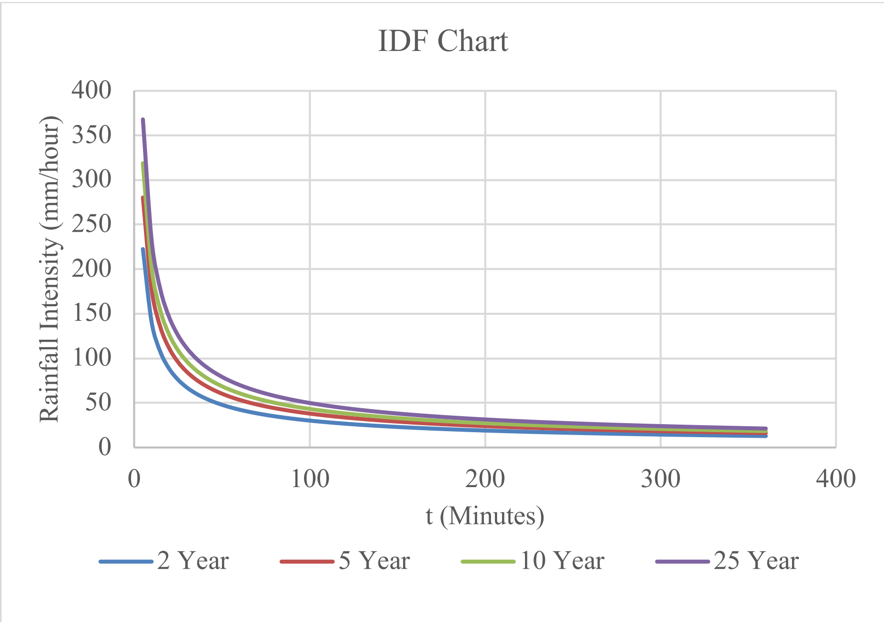

Figure 2. Intensity Duration Frequency

Analysis of Runoff Coefficients and Design Flood Discharge

The analysis of runoff coefficients requires land use data for the watershed, which is obtained through image interpretation and on screen digitization. Area delineation is performed using Google Earth; the data is then exported (as a KMZ file) and processed in ArcMap (ArcGIS). Next, digitization is performed for land use classification, and the area of each class is calculated using the Calculate Geometry feature. These areas are then used in the runoff coefficient calculation.

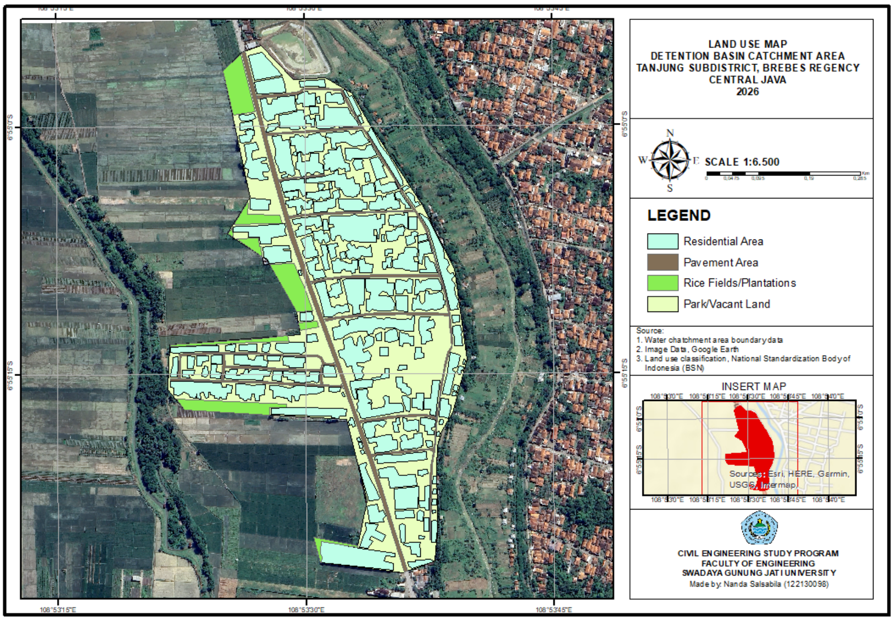

Figure 3. Land Use Map

Table 6 Runoff Coefficient

Land Use Types	Area (Ha)	C	Area x C
Paved Areas	1.85	0.95	1.75
Residental Areas	13.02	0.95	12.37
Rica Fields / Plantations	1.57	0.20	0.31
Parks / Vacant ands	10.76	0.25	2.69
Water Chatchment Area (∑ Area)	27.20		17.13
C = (∑ Area x C) / ∑ Area		0.630	

The results from the table above indicate that the runoff coefficient to be used for the subsequent analysis is 0,63.

A design flood discharge analysis was conducted to estimate the magnitude of the peak discharge that may occur during a specific return period at the study site. This calculation integrates the results of the previous hydrological analysis, namely rainfall intensity, runoff coefficient, and catchment area. Flood discharge calculations using the rational method for various return periods show a trend of gradually increasing capacity; for return periods ranging from 2 to 25 years. Specifically, this study's analysis focuses on the 25-year return period, which yields a flood discharge of 4,006 m^3/s.

Hydraulic Analysis and Detention Basin Performance

The analysis of detention basin capacity begins with the processing of topographic data in the form of the basin's design coordinates. The initial step involves inputting the basin's design data into Civil 3D to create a surface. It is known that the basin's base is at an elevation of +4.3 m and the basin's crest is at an elevation of +8 m. From this surface data, the data is exported to ArcGIS format for further analysis. Volume calculations are performed in stages for each specific elevation range, which serves as the basis for flood routing analysis. Based on the analysis results, the total volume of the basin at an elevation of +8.0 m is 26552,35 m³.

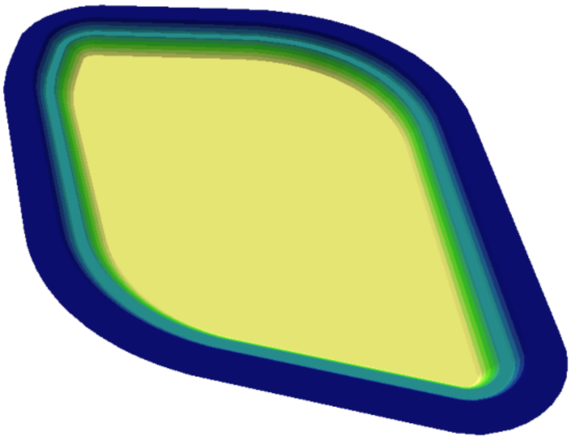

Figure 4. Detention Basin 3D

A flood routing analysis of the detention basin was conducted to examine the relationship between inflow, the volume of water stored in the basin, and outflow. The outflow analyzed in this study uses a sluice gate structure calculated based on the gate opening, where the water depth (h) is calculated as the difference between the water surface elevation and the outlet gate sill elevation.

In determining the discharge rate released through the sluice gate, a discharge coefficient (Cd) of 0,611 was used. This value is based on a reference discussing flood control gates, where it is a commonly used value (Kodoatie, 2021). Outflow is regulated through a 1 x 1 m square outlet. The outflow analysis is then divided into three scenarios of sluice gate opening variations: closed gate (a = 0 m), half-open gate (a = 0.5 m), and fully open gate (a = 1 m). The flood routing analysis is presented in the following table.

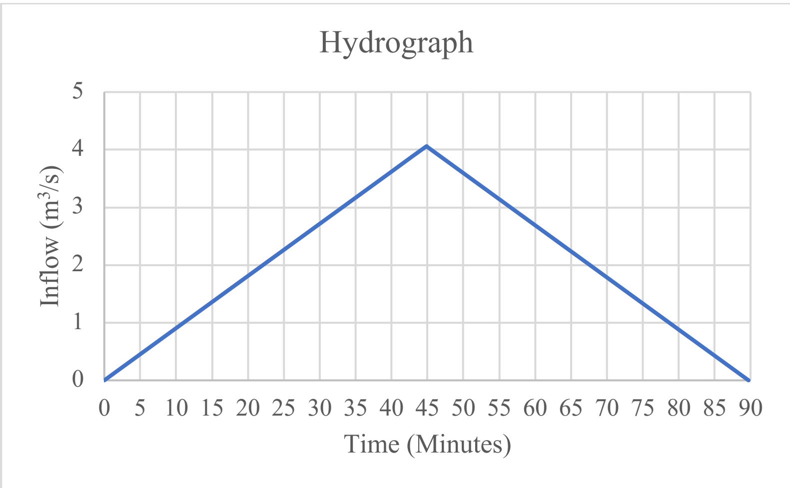

Figure 5. Triangular Hydrograph Q25

According to the graph, the discharge increases until it peaks at Tc, then gradually decreases after passing Tc until it reaches 2Tc.

The flood analysis results were used to analyze the relationship between time and the retention volume of the basin, water level elevation, and outflow. The results of this analysis were then presented in graph form to illustrate how the characteristics of the detention basin change over time.

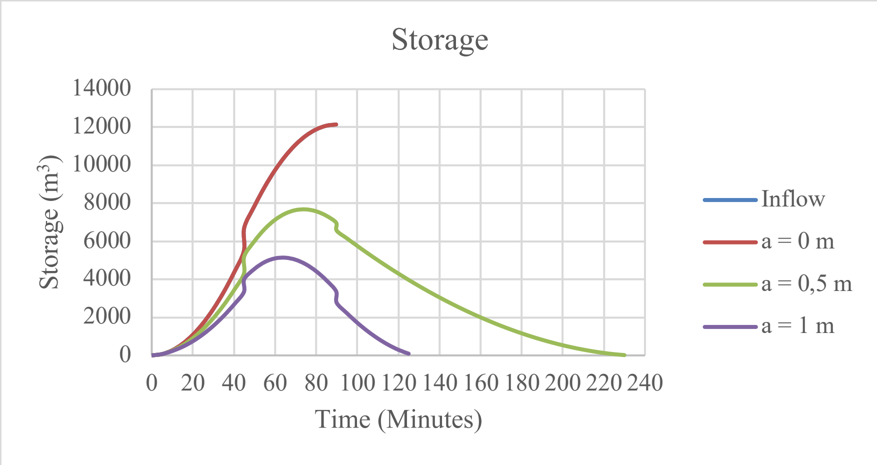

Figure 6. Basin Storage Over Time

The graph shows that the highest volume at the closed gate occurred at the 90th minute, with a basin water volume of 12144,636 m³; while the highest volume with the gate half-open occurred at the 75th minute, with a water volume in the basin of 7678,546 m³; and the highest volume with the gate fully open occurred at the 65th minute, with a water volume in the basin of 5138,120 m³.

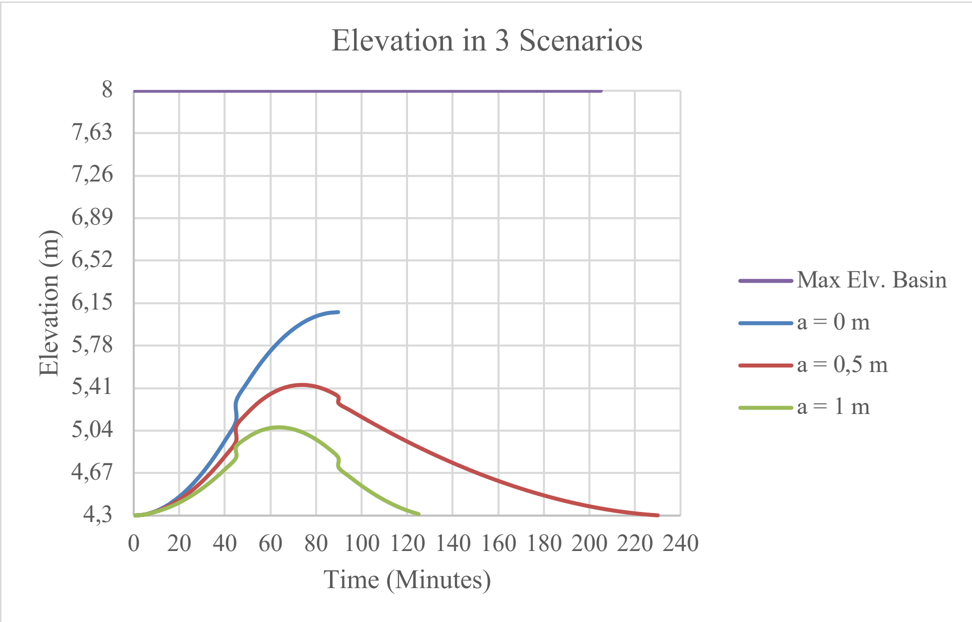

Figure 7. Water Level in the Basin Over Time

The elevation of the basin bottom is +4,3 m. The highest water level in the basin with the gate closed, recorded at the 90-minute mark, is +6,07 m; the highest water level in the basin with the gate half-open, recorded at the 75-minute mark, is +5,44 m; and the highest water level in the basin with the gate fully open, recorded at the 65-minute mark, is +5,07 m.

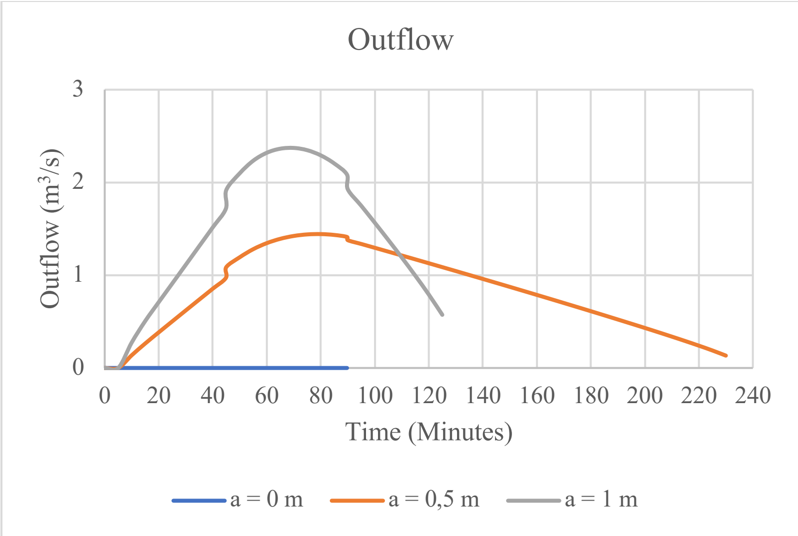

Figure 8. Outflow Over Time

The outflow graph shows that a closed gate produces no outflow; a half-open gate produces a maximum outflow of 1,443 m³/s at the 80th minute; and a fully open gate produces a maximum outflow of 2,372 m³/s at the 70th minute.

The graph shows that for a half-open gate, at the 230th minute, an outflow of 0,133 m³/s is obtained with a remaining storage volume of 15,621 m³; whereas for a fully open gate, at the 125th minute, an outflow of 0,573 m³/s is obtained with a remaining storage volume of 90,322 m³. This indicates that during that interval, outflow continues as long as there is still water volume available to be discharged from the detention basin.

In this study, the primary analysis of the detention basin's capacity was based on the design flow condition Q25, considering the operational scenario of the sluice gate at half-opening as the most optimal condition, the analysis results show that the reduction in flood discharge at half-opening reached 64.44%, which is greater than the 41.57% reduction at full opening. This condition was then used as the primary reference for evaluating the detention basin's performance in terms of flood control capability. Furthermore, to obtain a more comprehensive picture of the system's behavior under varying flow rates, additional analyses were conducted using several reference flow rates: Q2, Q5, Q10, and Q25.

The analyses of these Q2–Q25 flow rates were then used to identify trends in how the detention basin's response changes with increasing rainfall return periods. Additionally, this approach aims to determine to what extent the detention basin can still handle runoff under conditions ranging from lower to more extreme discharge levels. Thus, the results of this analysis serve as a comparison to the primary Q25 condition, thereby providing a more comprehensive picture of the detention basin's performance across various discharge scenarios. The following is a graph of the analysis results.

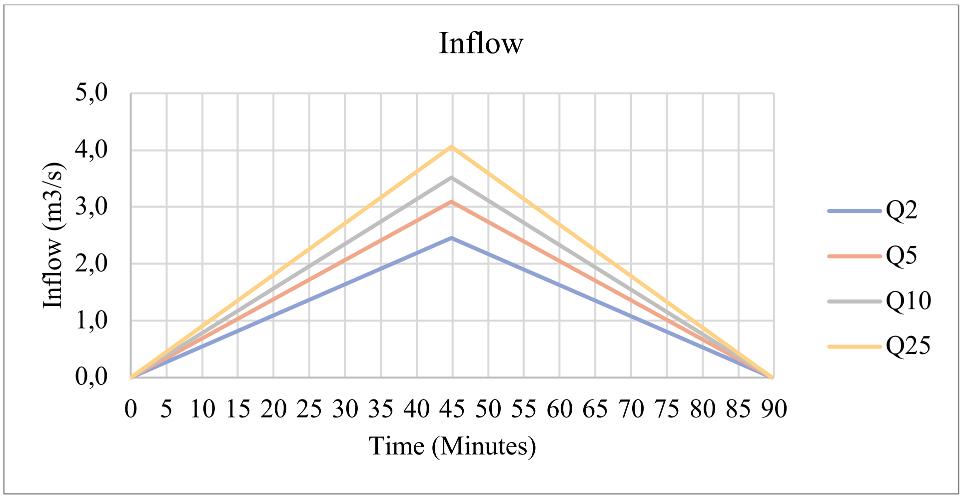

Figure 9. Summary of Inflows (Q2-Q25)

The graph shows that inflow discharge increases as the return period increases, with the highest value occurring at a 25-year return period.

Table 7 Summary of Freeboard on Closed Sluice Gate (Q2-Q25)

Return Period	**Elv. Max (a = 0 m)**	**Freeboard (a = 0 m)**	**Description**
(Year)	**(m)**	**(m)**	
Q2	5.39	2.61	Safe
Q5	5.66	2.34	Safe
Q10	5.84	2.16	Safe
Q25	6.07	1.93	Safe

Table 7 shows that, for a closed sluice gate, the elevation increases as the return period increases, with the highest value occurring at a 25-year return period.

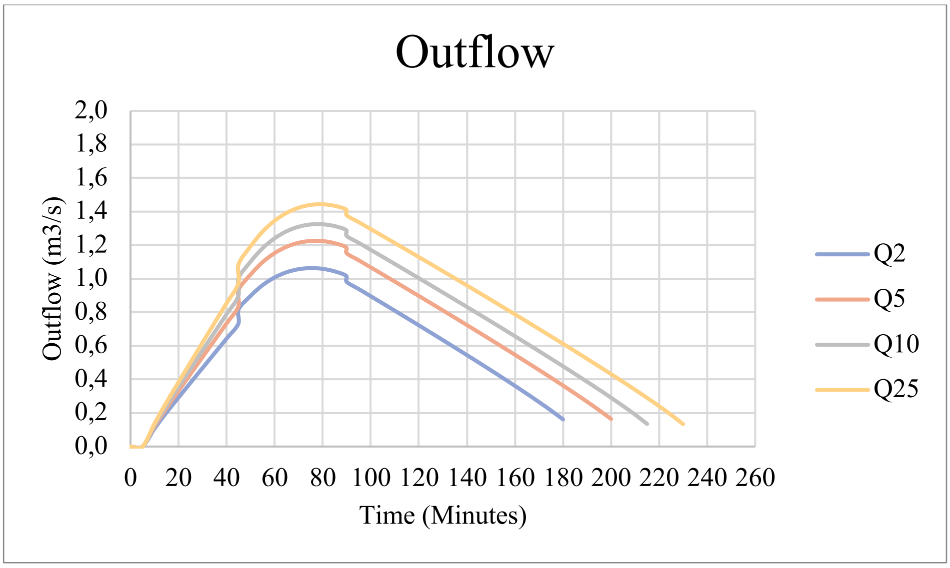

Figure 10. Summary of Outflows (Q2-Q25)

Analysis of the discharge summary graph (Q2–Q25) using an outflow condition with the gate half-open, as this is considered more effective in reducing flooding than a fully open gate. The graph shows changes in outflow discharge over time for each return period scenario. The outflow value increases as the inflow discharge increases, then reaches a maximum value before decreasing. This pattern occurs in all scenarios with varying magnitudes depending on the return period.

Table 8 Summary of Flood Reduction (Q2-Q25)

Return Period	**Qin**	**Qout (a = 0,5 m)**	**Reduction**
(year)	**(m3/s)**	**(m3/s)**	**(%)**
Q2	2.455	1.063	56.692
Q5	3.093	1.225	60.390
Q10	3.518	1.324	62.351
Q25	4.060	1.443	64.444

Table 8 shows the percentage reduction in flood discharge for various return periods (Q2 to Q25) with the gate half-open. The analysis results indicate that the flood reduction values range from approximately 56% to 64%. In general, there is a trend of increasing flood reduction values as the return period increases, although this increase is not particularly significant and remains relatively stable across scenarios.

CONCLUSION

Based on the analysis results, flood discharge is influenced by rainfall intensity and the size of the catchment area, where short-duration, high-intensity rainfall has the potential to generate significant runoff in a short period of time. Based on the rainfall analysis, the design flood discharge for a 25-year return period (Q25) was determined to be 4,060 m^3/s, occurring at a concentration time of 44,861 minutes. The detention basin has a storage capacity that remains safe for accommodating the runoff volume, as indicated by the available freeboard. At a 25-year return period, the inflow volume of 12144,64 m^3 results in a water depth of 1,77 m and leaves a remaining freeboard of 1,93 m. Analysis results for various return periods (Q2–Q25) indicate that the basin remains capable of accommodating flood discharge up to a 25-year return period. Additionally, the operation of the sluice gate affects the reservoir's performance, where a half-open gate reduces flood discharge by 64,4% and represents the most optimal operating condition compared to a fully open gate.

The operation of the sluice gate must be flexible to accommodate downstream conditions. During downstream flooding, the gates should be closed to prevent backflow, whereas during low water conditions, the gates can be fully opened to accelerate the draining of the reservoir. Under normal conditions or when inflow can still be controlled, the gates can be operated at a half-open position (0,5 m) because this has been shown to provide a greater percentage reduction in flood discharge compared to full opening. It is recommended to add a pump system to ensure that standing water after rain is effectively removed, thereby preventing the accumulation of stagnant water that could compromise sanitation and hygiene in the surrounding area.

REFERENCES

Amalia, M., Muchamad, B. N., Sutanti, M. P., Sofia, E., & Helda, N. (2023). Measuring the Use of Detention Basin as Flood Control System at Campus I Lambung Mangkurat University Banjarmasin, Indonesia. IOP Conference Series: Earth and Environmental Science, 1184(1). https://doi.org/10.1088/1755-1315/1184/1/012013

Arashi, F. B., Lestari Iskandar, A., Sarifah, F., Azril, M., Ramadhan, R., Daniswara, M. P., & Rahmadhani, F. (2024). Analisis Dampak Bencana Banjir terhadap Kondisi Sosial dan Ekonomi pada Masyarakat.

BNPB. (n.d.). Kompilasi data kejadian dan dampak bencana di Indonesia. Pusat Data, Informasi dan Komunikasi Kebencanaan. Diakses dari https://data.bnpb.go.id/dataset/data-bencana-indonesia

Gultom, N., Badaruddin., & Kadir, S. (2022). Analisis Debit Air di Daerah Tangkapan Air (DTA) Barabai Kabupaten Hulu Sungai Tengah. Jurnal Sylva Scienteae, (05)5.

Hasan, F., & Widyanto, B. E. (2025). Effektivitas Penerapan Kolam Detensi dan Sistem Pompa dalam Pengendalian Banjir di Kampung Rawa Bamban. JURNAL TEKNIK HIDRAULIK, 16(1), 39–48. https://doi.org/10.32679/jth.v16i1.825

Kodoatie, R. J. (2021). Rekayasa dan Manajemen Banjir Kota. Yogyakarta: Andi.

KP-04 Tahun 2013 tentang Pedoman Perencanaan Bangunan Pengendali Banjir

Marsidi, M. B., (2024). Simulasi Kolam Detensi Pengendali Banjir Kota Sengkang (Kasus Jl. Dahlia – Veteran).

Nursella, S., & Ariyanto, L. (2025). Evaluasi Fungsi dan Analisis Kapasitas Dimensi Saluran Drainase di Kawasan Permukiman Kota Bandar Lampung. Jurnal Konstruksi, 23(1), 211–223. https://doi.org/10.33364/konstruksi/v.23-1.2357

Pudyastuti, P. S., & Musthofa, R. A. (2020). Analysis of Maximum Daily Rainfall Distribution at Selected Rain Gauging Stations in Klaten Region In Years 2008-2018.

Sahir, S. H. (2021). Metodologi Penelitian. Yogyakarta: KMB Indonesia

Setiawan, H., Jalil, M., Enggi, M., Purwadi, F., Adios, C., Brata, A. W., Syaful Jufda, A., Studi, P., Geografi, P., Keguruan, F., Pendidikan, I., & Mulawarman, U. (2020). Analisis Penyebab Banjir Di Kota Samarinda. In Jurnal Geografi Gea (Vol. 20, Issue 1). https://ejournal.upi.edu/index.php/gea

SNI 2415:2016 tentang Tata Cara Perhitungan Debit Banjir Rencana

Soewarno. (2015). Klimatologi Pengukuran dan Pengolahan Data Curah Hujan, Contoh Aplikasi Hidrologi dalam Pengelolaan Sumber Daya Air. Yogyakarta: Graha Ilmu.

Suripin. (2004). Sistem Drainase Perkotaan yang Berkelanjutan. Yogyakarta: Andi.

Sutiyono, Y. (2021). Analisis Perubahan Tata Guna Lahan Terhadap Debit Banjir Di DAS Bengawan Solo Hulu (Analysis of Land Use Change on Flood Discharge in Bengawan Solo Upstream Watershed).

Triatmojo. (2008). Hidrologi Terapan. Yogyakarta: Beta Offset.

Widyawati., Yuniarti, D., & Rito Goejantoro, dan. (2020). Analisis Distribusi Frekuensi dan Periode Ulang Hujan (Studi Kasus: Curah Hujan Kecamatan Long Iram Kabupaten Kutai Barat Tahun 2013-2017) Frequency Distribution Analysis and Rain Return Period (Case Study: Rainfall data of Long Iram Sub-Distric, West Kutai Distric in 2013 to 2017). Jurnal Eksponensial, 11(1).

Wigati, R., Putu Purnaditya, N., Ujianto, R., Sabri, F., Kamal, S., & Zulfa, G. F. (2021). SI DUDUNG (Sistem Drainase Hijau dan Unggul) Sebagai Solusi Banjir Kota Serang Banten. In Jurnal Teknik Sipil (Vol. 10, Issue 1). https://www.google.com/maps/place/

Yandri, N. (2021). Analisis Efektivitas Kolam Detensi Danau Cimpago Untuk Pengendalian Banjir.