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The Effects of Spillway Width, Inflow-Outflow Discharge, and Flow Elevation on Weir Crest

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Abstract

The purpose of this study was to analyze the effect of spillway width on flow elevation above weir crest based on the flood discharge design for the Probable Maximum Flood (PMF) return period using flood routing hydrologically at the Cacaban Dam. The rainfall Probable Maximum Precipitation (PMP) design used the Hershfield equation. The design of the flood discharge analysis of QPMF used the Nakayasu Synthetic Unit Hydrograph (HSS). Flood routing used hydrologic routing method. The Cacaban Dam is located in Jati Village, Kedung Banteng District, Tegal Regency, Central Java Province. The results of the research data analysis showed that bigger spillway crest widths led to lower flow heights above the spillway crest, and bigger outflows. Thus, if a large storage volume of the reservoir is intended, then the width of the spillway crest must be reduced. Otherwise, the width of the spillway crest must be increased. In relation to flood control in Tegal Regency, reducing the spillway crest width is preferable.

Keywords: Flood Routing; Inflow and Outflow Discharge; Flow Elevation; Spillway Crest.

1. Introduction

Reservoirs, in the general sense, are places on the ground that are intended to store or retain water in the event of excess water in the rainy season. The abundant water is then used for agricultural and other purposes during the dry season. The reservoir serves as a water source. In addition, it also serves as a controller of floods and droughts and as a means for recharge to increase the availability of groundwater. Reservoirs also provide benefits for fishing, tourism, and other activities. Therefore, their presence, if managed properly, will be able to provide added value to the surrounding area.

The Cacaban Dam is geographically located between 109° 11' 28" East and 109° 14' 58" East and between 7° 1' 31" South and 7° 4' 18" South. It is located in Jati Village, Kedung Banteng District, Tegal Regency, Central Java Province, bordering Brebes Regency in the west and east and Pemalang Regency in the north. It is bordered by Tegal City and the Java Sea and by Brebes and Banyumas Regencies in the south [1]. The Cacaban Reservoir has a catchment area of 6,792 ha. The topography of the Cacaban Dam is a hill with an altitude of 85 m to 600 m above sea level. This dam is a homogeneous soil pile dam with a height of 38 meters and a length of 168 meters. The elevation of the peak of the dam is +80.50 m, the normal water level is +77.5 m, and the flood water level is +78.75 m. The total reservoir volume is

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74.82 million m^3, which is used to serve an irrigation area of 17,481 hectares.

At the Cacaban Reservoir or Dam, according to Anggara & Sundary (2017) [2], at an elevation of normal water level and at an elevation of +77.50 m, the reservoir volume was 55.68 million m^3, and at an elevation of dead reservoir EL of +63.00 m, the reservoir volume was 0.50 million m^3. In other words, the effective volume of the reservoir in 2016 was 55.18 million m^3. There was a difference in volume of 0.67 million cubic meters compared to the measurement results in 2012.

The volume of the Cacaban Dam has decreased every year. Between 2012 and 2016, there was a volume reduction of 0.67 million m^3. In 2020, it is predicted that the volume decline will be much greater than that figure. The decrease in the volume of the Cacaban Dam is assumed to be one of the important parameters of flooding in Tegal Regency. Therefore, accurate information about the elevation above the crest is required so that the renovation of the Cacaban Dam is right on target. Information on the parameters of the effective width and height of the flow elevation above the overflow crest of the dam is closely related to the amount of budget that will be used in dam renovation activities.

The objectives of this research were (1) to analyze the effect of spillway width on inflow and outflow discharges based on the flood discharge design for the PMF return period using flood routing hydrologically at the Cacaban Dam, and (2) to analyze the effect of spillway width on flow elevation above the weir crest based on the flood discharge design for the PMF return period using flood routing hydrologically at the Cacaban Dam.

2. Method

This research used a survey research method.. The rainfall data used were on the annual maximum daily rain. The rainfall data were obtained from the Tegal Regency Agriculture Office. The rain stations used for hydrological calculations were in Sirampok (109° 11’ 0.276" E; 7° 0’ 17.425" S), Lebaksiu (109° 8’ 23.879" E; 7° 3’ 26.368" S), and Jatinegara (109° 15’ 0.176" E; 7° 3’ 50.78"). The length of time of the daily rainfall data was 10 years, from 2008 to 2017. The rainfall data were used to analyze the design rainfall, design flood discharge, and flood hydrograph, which were then used for the input-output analysis of flooding at the Cacaban Dam spillway's crest [1].

Software and hardware were used in this research. Excel, Arc view GIS, and Auto CAD were the software used in the research. A computer, a camera, Android, and GPS were used as the research hardware. The source of this research was a 1:50000 scale topographic map obtained from the most recent Google map in 2018. The Thiessen polygon was used to identify the distribution of the catchment area using topographic maps at a resolution of 1:50000. Arc view GIS and AutoCAD software was used to analyze the Cacaban watershed and sub-watershed [3].

Flood routing reservoirs were used in hydrologic routing based on the continuity equation below [4-6]:

$$I - Q = \frac{d_S}{d_t} \quad (1)$$

where I is the average inflow discharge in a small time interval d_t, Q is the average outflow discharge in the same time interval (m^3/s), d_S is the corresponding change in the storage of the reservoir during the same time interval (m^3), and d_t is the flood routing period (s).

While d_t is changed to Δt, I_1 and I_2 can be known from the hydrograph of discharge into the reservoir. S represents the storage of the reservoir at the beginning of the routing period measured from the reference line of expenditure facilities (spillway weir or axis tunnel outlet). The flood routing equation according to Hossain (2015), Ionescu & Nistoran (2019) and Sutapa (2019) [4-6]:

$$I\frac{I_1+I_2}{2} + \left(\frac{S_1}{\Delta t} - \frac{Q_1}{2}\right) = \frac{S_2}{\Delta t} + \frac{Q_2}{2} \quad (2)$$

if $\frac{S_1}{\Delta t} - \frac{Q_1}{2} = \psi_1$ and $\frac{S_2}{\Delta t} - \frac{Q_2}{2} = \phi_2$.

Thus, Equation 2 can also be written as:

$$\frac{I_1+I_2}{2} + \psi_1 = \phi_2, \quad (3)$$

where I_1 is the incoming discharge whose position in the calculation table is above the discharge to be found (m^3/s), I_2 is the incoming discharge to be found (m^3/s), ψ_1 is the conditions at the start of routing, ϕ_2 is the conditions at the end of routing, Δt is the flood routing period (seconds, hours, or days), and S is the large storage reservoir (m^3). Q is the outflow at the beginning of the routing period. If its expenditure is spillway, then the equation used is as presented below [7-10]:

$$Q = CBH^{3/2} \quad (4)$$

where C is the discharge coefficient for spillway (1.7-2.2 $m^{1/2}/s$), B is the spillway weir width (m), and H is the high energy above the spillway weir (m).

The Cacaban Dam is of the homogeneous soil fill type, with a peak length of 168 m, a peak width of 6.0 m, and a peak elevation of +80.50 m. The spillway specification of Cacaban Dam is as follows: (1) doorless ogee type, (2) crest elevation of +77.50 m, (3) crest width of 58 m, and (4) spillway width of 16 m. Figure 1 shows the construction of the spillway and chute spillway of the Cacaban Dam [3].

Figure 1. Construction of the spillway and chute spillway of the Cacaban Dam

At normal water level elevation at +77.50 m, the reservoir volume was 55.51 million m^3, and at the elevation of dead storage at EL of +63.00 m the reservoir volume was 0.50 million m^3. In other words, the effective volume of the reservoir in 2016 was 55.51 million m^3.There was a volume difference of 0.84 million cubic meters when compared to the measurement results in 2012 (56.35 million m^3). Table 1 shows the reservoir capacity of the Cacaban Dam in 2016.

Table 1. Reservoir capacity of the Cacaban Dam in 2016 [2]

Number	Elevation (m)	Storage (m^3)	Number	Elevation (m)	Storage (m^3)
1	77.5	55514687.37	12	78.6	66136322.58
2	77.6	56480290.57	13	78.7	67101925.78
3	77.7	57445893.77	14	78.8	68067528.98
4	77.8	58411496.97	15	78.9	69033132.18
5	77.9	59377100.17	16	79	69998735.38
6	78	60342703.37	17	79.1	70964338.58
7	78.1	61308306.57	18	79.2	71929941.78
8	78.2	62273909.77	19	79.3	72895544.98
9	78.3	63239512.97	20	79.4	73861148.18
10	78.4	64205116.17	21	79.5	74826751.38
11	78.5	65170719.38			

The steps of this research were as follows (1) conducting a review of relevant previous research, followed by formulating the problem; (2) collecting daily rainfall data; (3) performing an analysis of the rainfall data; (4) examining the distribution of the rainfall data; (5) analyzing the planned rainfall; (6) determining the design flood discharge using the Nakayasu Synthetic Unit Hydrograph; (7) determining the discharge value of the Q PMF flood plan; (8) analyzing the flood tracking due to the Q PMF; (9) analyzing the flow elevation above the spillway crest; (10) comparing the flow elevation above the calculated spillway crest with the flow elevation above the existing spillway crest (if h analysis < h existing, then the iteration process was stopped); and (11) compiling a graph of inflow and outflow discharge on the Q PMF. Figure 2 shows the research flow chart.

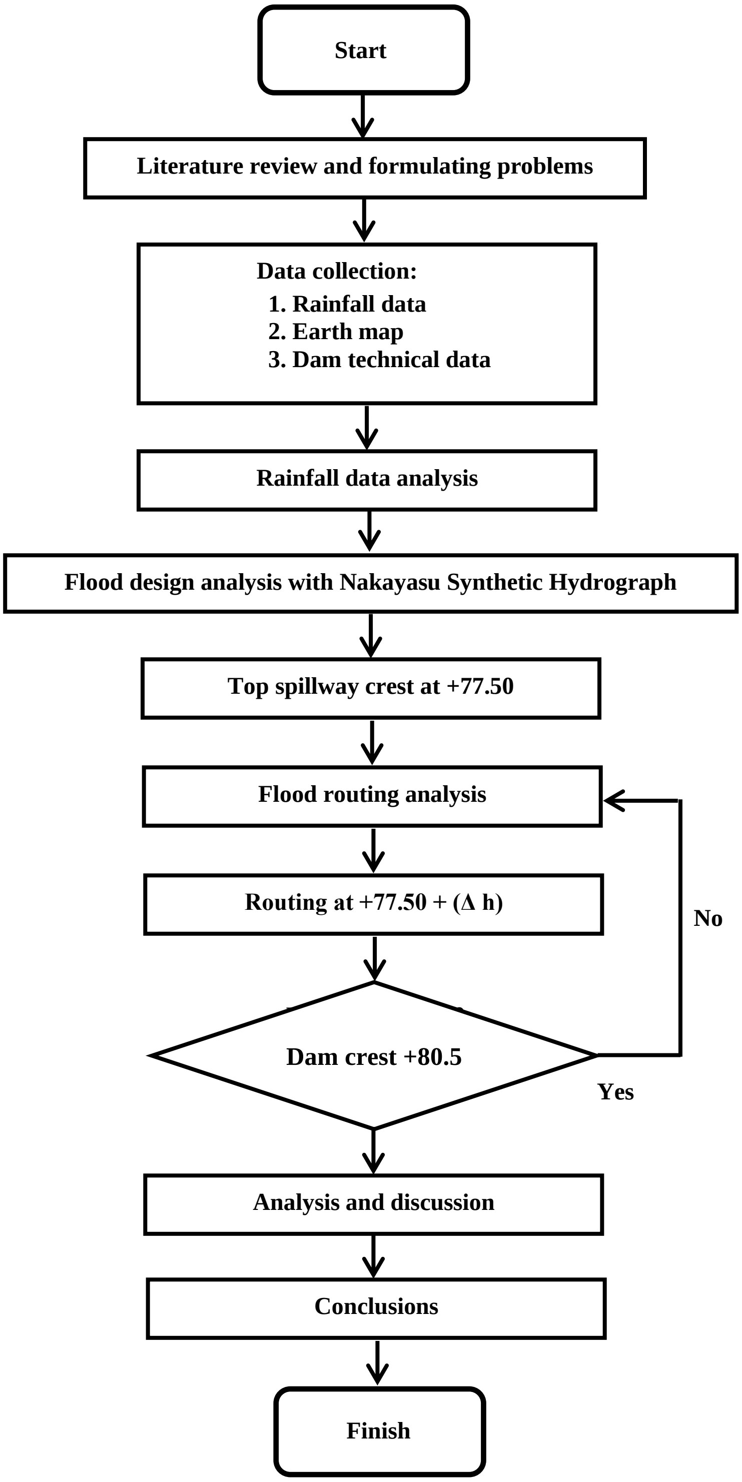

Figure 2. Research flow chart

3. Results and Discussion

3.1. Regional Rainfall Distribution

The rainfall data available were historical data. Thus, the hydrological calculation was based on the data at rain stations affecting the Cacaban catchment area. The rain stations used for the hydrological calculations were those in Sirampok, Lebaksiu, and Jatinegara. The length of time of the three stations' data were 10 years. The rainfall data used were annual maximum daily rainfall. Below is Figure 3: Cacaban Catchment Area [3].

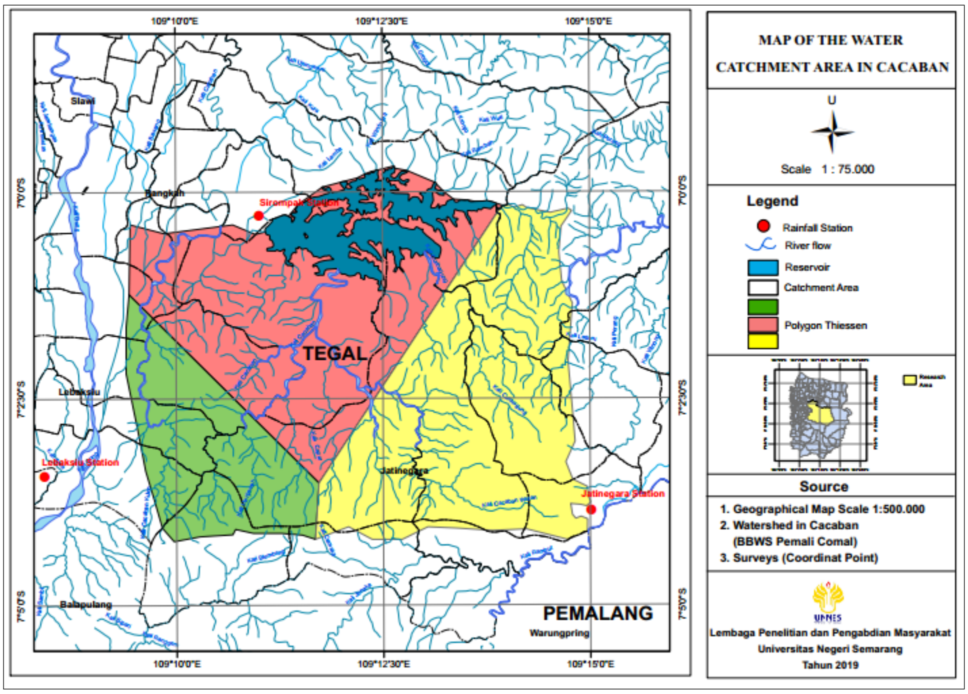

Figure 3. Map of the water catchment area in Cacaban

The rainfall data obtained were the point rainfall data of a station. Therefore, an analysis was required to process the data into regional rainfall data. This research used the point rainfall data at the three stations, so the regional rainfall data were processed based on the rainfall data at the three stations. The analysis used the Thiessen polygon method. Below is provided Table 2 on the distribution of the catchment area using the Thiessen polygon method.

Table 2. Thiessen Polygon Coefficient

Number	Station	Catchment area or A_i (Km^2)	Thiessen's Coefficient (%)
1.	Sirampok	27.667	41.711
2.	Lebaksiu	12.707	19.157
3.	Jatinegara	25.955	39.130
	Total	66.329	100

The rainfall distribution of each region was obtained by an analysis using the Thiessen polygon method, considering the factors involved in the Thiessen polygon. The results of the calculation of maximum regional daily rainfall are provided in Table 3.

Table 3. Maximum regional daily rainfall at Sirampok, Lebaksiu, and Jatinegara Stations using the Thiessen polygon method (mm)

Year	January	February	March	April	May	June	July	August	September	October	November	December
2008	64.501	44.118	68.943	60.807	22.368	21.117	0.000	28.043	8.342	50.685	60.773	66.631
2009	81.429	76.713	52.462	40.072	40.803	30.480	5.870	0.000	24.314	53.512	68.016	80.762
2010	65.239	96.720	92.777	57.046	52.139	95.013	49.114	55.226	54.783	59.497	58.131	73.720
2011	74.910	111.752	65.709	61.319	62.576	22.132	31.999	0.000	0.000	51.111	57.750	59.788
2012	97.131	85.446	45.598	49.664	30.329	22.126	18.760	0.000	0.000	32.147	47.489	157.535
2013	102.883	50.719	49.309	62.660	21.026	60.222	90.894	29.918	23.765	24.045	18.622	68.043
2014	50.894	122.276	84.218	72.046	46.817	83.250	53.515	16.764	0.000	8.738	76.844	67.257
2015	92.064	107.295	82.963	74.668	61.455	21.379	0.000	0.000	0.000	0.000	61.526	74.776
2016	40.023	111.752	82.781	61.319	39.561	92.262	36.667	15.813	77.035	33.163	53.291	86.535
2017	104.764	90.484	116.091	99.178	53.337	44.828	13.766	2.503	37.908	33.535	47.875	66.963

3.2. Rainfall Design with 20, 50, 100, 1000 Years and PMP Return Periods

The maximum rainfall for a given return period was determined using design rainfall analysis, which was then employed in the design discharge calculation. The study included return periods 20, 50, 100, and 1000 years as well as Probable Maximum Precipitation (PMP return periods. The method for calculating rainfall was based on statistics or distribution methods of average daily rainfall in the catchment area. The Log Pearson Type III distribution was employed to establish the design rainfall in this study [11]. PMP was statistically analyzed using the Hershfield equation [12-14]. Figure 4 shows a map of the catchment area of the Cacaban Dam.

Figure 4. Map of the water catchment area in Cacaban

Table 4 shows that the design rainfall was as follows: 147.97 mm in the return period of 20 years; 161.31 mm in return period of 50 years; 169.37 mm in return period of 100 years; 192.96 mm in return period of 1000 years; and 588.99 mm in the PMP. Rainfall (R24/daily) of 147.97 mm, 161.31 mm 169.37 mm, 192.96 mm, and 588.99 mm are included in the category of heavy to very heavy [15].

Table 4. High precipitation for the return periods of 20, 50, 100 years using the Log Pearson Type III distribution and PMP distribution

T (year)	k	Log X_T (mm)	X_T (mm)
20	1.514	2.170	147.97
50	1.898	2.207	161.31
100	2.115	2.228	169.37
1000	2.695	2.285	192.96
PMP	-	-	588.99

X is the observational variation value, X_T is the expected X variant value occurring in the return period of T, and k from the table is a function of the return period and the coefficient of variation [16].

3.3. Hydrograph of the Flood Design in the 20, 50, 1000 Years and PMF Return Periods

The design flood discharge of the Cacaban Reservoir was calculated using the design rainfall calculation as a commonly used hydrological approach. The Nakayasu Synthetic Unit Hydrograph was used to calculate the design flood discharge (HSS). The equation for the Nakayasu Synthetic Unit Hydrograph (HSS) is as follows [17-19]:

$$Q_P = \frac{AR_o}{3.6\,(0.3t_P + T_{0.3})} \quad (5)$$

where Q_P is the flood discharge peak (m^3/s), A is the catchment area of Cacaban reservoir (km^2), R_o is the rain unit (mm), T_P is the time lag from the beginning of the rain to the peak of the flood (hour), and $T_{0,3}$ is the time required by the discharge to descend from the peak discharge to 0.3 times the peak discharge (hour).

Below is presented Figure 5 on the hydrograph of the flooding design for the 20, 50, 100, and 1000 years and PMF return periods.

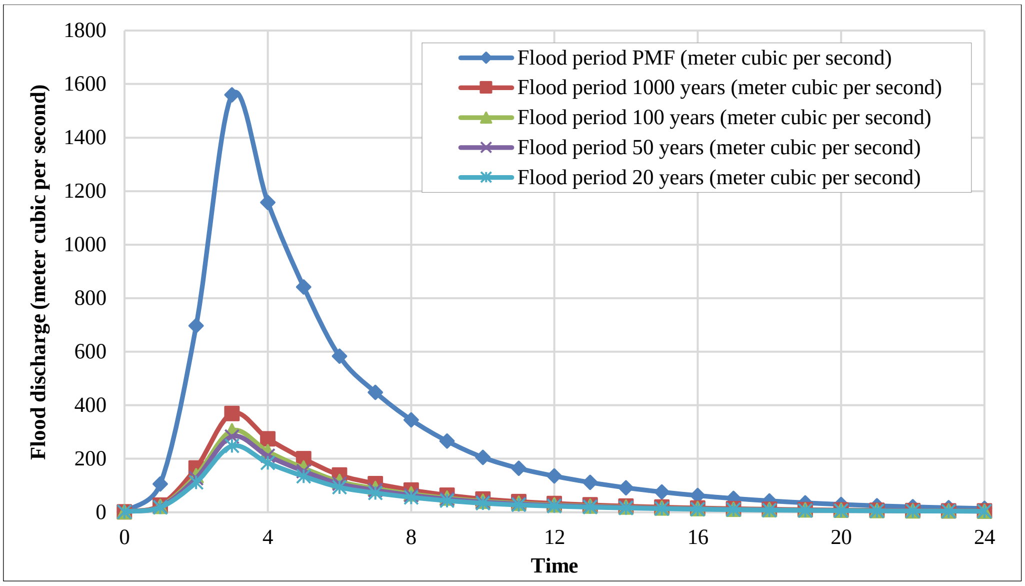

Figure 5. The hydrograph of flood design for the 20, 50, 100, 1000 years and PMF return periods

Figure 5 shows that (1) the design flood discharge for the return period of 20 years is 247.750 m^3/s, (2) the design flood discharge for the return period of 50 years is 283.343 m^3/s, (3) the design flood discharge for the return period of 100 years is 305.116 m^3/s, (4) the design flood discharge for the return period of 1000 years is 369.775 m^3/s, and (5) the design flood discharge for the PMF return period is 1,559.429 m^3/s.

3.4. Flood Routing Based on the PMF Flood Discharge Return Period

The PMF return period flood routing was conducted through several stages [20]: (1) arranging Table 5 on the relationship between water reservoir elevation, storage, and discharge (ψ); (2) determining the regression equation between water reservoir elevation and storage and between water reservoir elevation and discharge (ψ); (3) compiling Table 6; and (4) creating a chart of flood routing through a spillway (inflow and outflow discharge chart). The stages above follow the notion that flood routing through a spillway is aimed to figure out the level of runoff when the flood discharge passes the spillway. The flood routing result is to be used as a basis for determining whether overtopping occurred at the dam or not [21].

Table 5. The relationship between the water surface of the reservoir, the storage, and the discharge (ψ) at the Cacaban Dam (QPMF)

Number	Elevation (h) (m)	H (m)	Storage (S) (m^3)	S (m^3/s)	Discharge (I) (m^3/s	I/2 (m^3/s)	ψ (psi) (m3/s)	φ (phi) (m^3/s)
(1)	(2)	(3)	(4)	(5)/3600	(6)	(7)	(8)	(9)
1	77.50	0	55514687.37	15420.746	0.000	0.000	15420.746	15420.746
2	77.60	0.1	56480290.57	15688.970	2.384	1.192	15687.777	15690.162
3	77.70	0.2	57445893.77	15957.193	6.744	3.372	15953.821	15960.565
4	77.80	0.3	58411496.97	16225.416	12.389	6.195	16219.221	16231.611
5	77.90	0.4	59377100.17	16493.639	19.075	9.537	16484.102	16503.176
6	78.00	0.5	60342703.37	16761.862	26.658	13.329	16748.533	16775.191
7	78.10	0.6	61308306.57	17030.085	35.043	17.521	17012.564	17047.607
8	78.20	0.7	62273909.77	17298.308	44.159	22.079	17276.229	17320.388
9	78.30	0.8	63239512.97	17566.531	53.952	26.976	17539.555	17593.507
10	78.40	0.9	64205116.17	17834.754	64.378	32.189	17802.566	17866.943
11	78.50	1	65170719.38	18102.978	75.400	37.700	18065.278	18140.678
12	78.60	1.1	66136322.58	18371.201	86.988	43.494	18327.707	18414.695
13	78.70	1.2	67101925.78	18639.424	99.116	49.558	18589.866	18688.982
14	78.80	1.3	68067528.98	18907.647	111.760	55.880	18851.767	18963.527
15	78.90	1.4	69033132.18	19175.870	124.900	62.450	19113.420	19238.320
16	79.00	1.5	69998735.38	19444.093	138.519	69.259	19374.834	19513.352
17	79.10	1.6	70964338.58	19712.316	152.599	76.299	19636.017	19788.616
18	79.20	1.7	71929941.78	19980.539	167.126	83.563	19896.976	20064.103
19	79.30	1.8	72895544.98	20248.762	182.087	91.044	20157.719	20339.806
20	79.40	1.9	73861148.18	20516.986	197.470	98.735	20418.250	20615.721
21	79.50	2	74826751.38	20785.209	213.263	106.632	20678.577	20891.840

Table 6. Analysis of flood routing through spillway of PMF discharge

Time (hour)	Inflow discharge (I_n) (m^3/s)	$(I_n+I_{n+1})/2$ (m^3/s)	ψ (psi) (m^3/s)	φ (phi) (m^3/s)	Outflow discharge (Q) (m^3/s)	H (m)	Elevation (h) (m)
(1)	(2)	(3)	(4)= equal (b)	(5)	(6)	(7)	(8)= equal (a)
0	1.896	0.948	15431.000	15,431.948	0.000	0.000	77.500
1	106.388	53.194	15531.636	15,584.830	0.869	0.038	77.538
2	696.757	348.378	16189.901	16,538.279	18.005	0.289	77.789
3	1,559.429	779.715	17647.990	18,427.704	89.901	0.844	78.344
4	1,158.315	579.158	18658.641	19,237.799	157.924	1.228	78.728
5	842.227	421.113	19305.948	19,727.061	207.739	1.475	78.975
6	582.833	291.417	19660.763	19,952.180	236.915	1.610	79.110
7	448.734	224.367	19861.130	20,085.497	253.947	1.686	79.186
8	345.589	172.795	19947.818	20,120.612	261.438	1.719	79.219
9	266.254	133.127	19952.374	20,085.501	261.833	1.721	79.221
10	205.232	102.616	19898.832	20,001.448	257.196	1.700	79.200
11	164.568	82.284	19811.212	19,893.496	249.667	1.667	79.167
12	135.503	67.751	19703.219	19,770.971	240.491	1.626	79.126
13	111.631	55.815	19581.325	19,637.141	230.273	1.580	79.080
14	92.024	46.012	19450.551	19,496.563	219.475	1.530	79.030
15	75.921	37.960	19314.757	19,352.718	208.448	1.478	78.978
16	62.694	31.347	19176.884	19,208.231	197.447	1.426	78.926
17	51.831	25.916	19039.141	19,065.056	186.657	1.373	78.873
18	42.909	21.455	18903.164	18,924.619	176.206	1.321	78.821
19	35.581	17.791	18770.143	18,787.933	166.177	1.271	78.771
20	29.563	14.781	18640.914	18,655.695	156.624	1.222	78.722
21	24.619	12.310	18516.046	18,528.355	147.575	1.174	78.674
22	20.559	10.280	18395.897	18,406.177	139.038	1.128	78.628
23	17.225	8.612	18280.670	18,289.282	131.012	1.085	78.585
24	14.486	7.243	18170.443	18,177.686	123.485	1.043	78.543

The coefficient of the relationship between the storage volume (S) and elevation (h) of the water reservoir obtained using the regression equation $h = -2x10^{-27}S^2 + 1x10^{-7}S + 77.5$ (a) was 1.0 (r = 1.0). Similarly the coefficient of the relationship between the elevation (h) and Psi (ψ) of the water reservoir obtained using the regression equation $\psi = 2627.6xh - 188208$ (b) was 1 (r = 1).

The hydrograph of inflow flood discharge for the PMF return period and the outflow discharge is shown in Figure 6. According to Figure 6, the peak outflow discharge of 261.833 m^3/s (spillway width of 58 m) was found at the elevation of +79.221 m. It can be said that the flood discharge for the PMF return period did not result in overtopping because the top of the dam was at an elevation of +80.50 m. Meanwhile, the hydrograph of inflow flood discharge for the PMF return period and the outflow discharge is shown in Figure 6, according to which the peak inflow discharge of 1,559.429 m^3/s was reduced to 261.833 m^3/s (outflow). This was due to the reservoir storage and spillway capacity. Thus, the Cacaban Reservoir could accommodate or store flood discharge of 1,297.596 m^3/s.

Figure 6 shows the hydrograph of the inflow flood discharge and outflow discharge for the PMF return period, with the highest outflow discharge being 261.833 m^3/s (spillway width of 58 m) at a height of +79.221 m. As a result, because the dam peak height is +80.50 m, the flood discharge for the PMF return period did not result in runoff. Figure 6 shows the inflow flood hydrograph for the PMF return period and the outflow discharge, with the peak inflow discharge lowered from 1,559.429 m^3/s to 261.833 m^3/s (outflow). This is related to the storage capacity of the reservoir and the capacity of the spillway. As a result, the Cacaban Reservoir can accommodate or store 1,297.596 m^3/s of flood runoff.

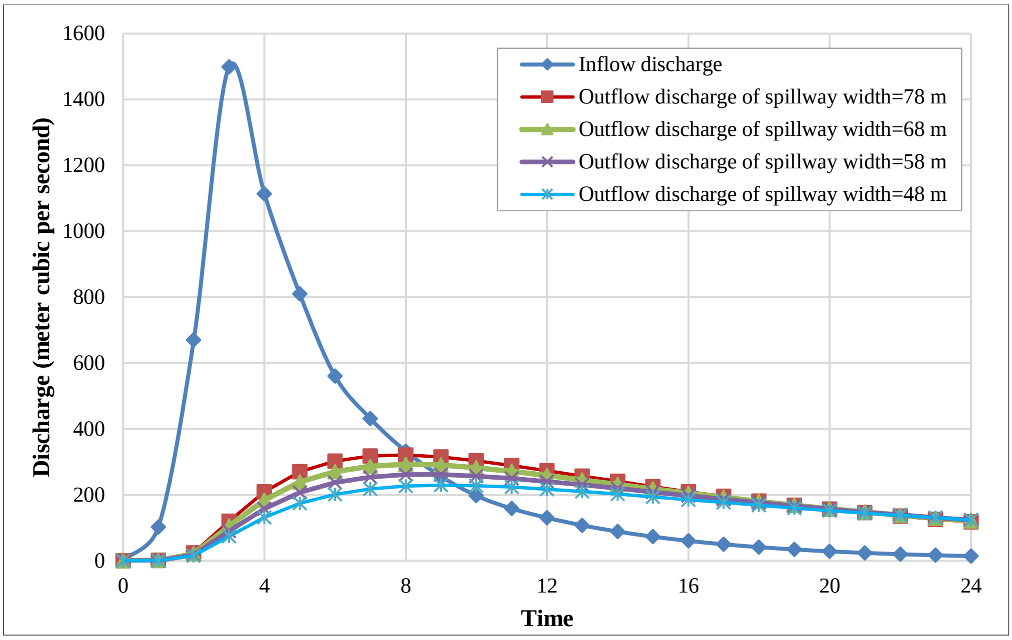

Figure 6. Graph of flood routing through the spillway for the PMF return period (inflow and outflow discharge)

3.5. The Effects of Spillway Width, Inflow-Outflow Discharge, and Flow Elevation on Weir Crest

Table 7 shows that an increase in the peak crest width of the spillway would cause the flow through it bigger while the level of pressure over decreased. The inflow flood discharge to the dam was channeled to the downstream section through the spillway, in which case the volume within would be reduced. In other words, the duration of the flooding would be prolonged, followed by a stagnant reservoir volume and lowered flood peak. This was because of the temporary addition of flood discharge storage time to the reservoir. Therefore, the reservoir only had storage water in a small volume within, and it would not suffice to lower the flood discharge peak. In this case, the outflow volume would approach the inflow volume. The smaller the volume of water temporarily contained by the reservoir the lower the pressure over the spillway crest.

Table 7. I_{max}, elevation, Q_{max}, and new dam height values due to flooding of the PMF return period design

Number	Spillway width (m)	Maximum inflow discharge or I_{max} (m^3/s)	High flow above weir (H) (m)	Elevation (h) (m)	Maximum outflow discharge or Q_{max} (m^3/s)	New dam height Freeboard=2 m [20]
1.	58	1,559.429	1.721	79.221	261.833	+81.221
2.	68	1,559.429	1.667	79.167	292.821	+81.167
3.	78	1,559.429	1.618	79.118	321.059	+81.118
4.	48	1,559.429	1.787	79.287	229.349	+81.287

The findings of this study are pertinent to those of Mediero et al. (2010) [22], who discovered that the width or length of the spillway crest is related to the elevation of the flow above the spillway crest, which is related to the size of the maximum outflow discharge in a sequential manner. From a hydrological standpoint, the relationship between the three parameters listed above is useful for determining the level of dam safety. Furthermore, the relationship between the three parameters indicated above can be used as one of the criteria evaluated in flood management at the dam's downstream location, which in this case is the City and Regency of Tegal. The findings of Mediero et al. (2010) [22], which are supported by Volpi et al. (2018) [23], show that the spillway crest dimension is one of the most critical elements in reducing flood peaks or increasing and decreasing reservoir water storage capacity.

4. Conclusion

This section outlines the research conclusions that may be taken from the research and discussion above. The spillway crest width of 58 m indicates that the inflow discharge was 1,559.429 m^3/s and the outflow discharge was 261.833 m^3/s during the PMF return period, with a flow elevation of +79.221 m; there was no overtopping because the dam's top height was +80.50 m. In addition, the Cacaban Reservoir could lower flood discharge by 1,297.596 m^3/s (83.21%). The spillway crest width of 48 m indicates that the inflow discharge was 1,559.429 m^3/s and the outflow discharge was 229.349 m^3/s during the PMF return period, with a flow elevation of +79.287 m; there was no overtopping because the dam's top height was +80.50 m. In addition, the Cacaban Reservoir could reduce flood discharge by 1,330.08

m^3/s or 85.29%. As a result, the flow elevation above the spillway crest decreased as the breadth of the spillway crest increased, while the maximum outflow discharge increased. Thus, if a large storage volume in the reservoir is required, the width of the spillway crest must be reduced; alternatively, the spillway crest must be increased. The height of the flow above the spillway crest is related to the size of the maximum outflow discharge, which is related to the breadth or length of the spillway crest. From a hydrological standpoint, the relationship between the three parameters listed above is useful for determining the level of dam safety. Furthermore, one of the criteria examined in flood control downstream of the Cacaban Dam, which in this case is flood control in the Regency of Tegal, is the relationship between the three parameters indicated above.

5. Declarations

5.1. Author Contributions

Conceptualization, Y.S. and K.S.U.; methodology, Y.S. and K.S.U.; software, Y.S., K.S.U. and N.T.; validation, Y.S., K.S.U. and N.T.; formal analysis, Y.S. and K.S.U.; writing—original draft preparation, Y.S.; writing—review and editing, Y.S.; visualization, N.T.; supervision, K.S.U.; project administration, Y.S.; funding acquisition, Y.S. All authors have read and agreed to the published version of the manuscript.

5.2. Data Availability Statement

The data presented in this research were collected from the Regional Planning, Development, Research, and Development Agency (BAPPEDA) of Tegal Regency and the Pemali Juana River Basin Center (BBWS) of Semarang City. The data from the aforementioned agencies were complemented with the rainfall data from the Public Work Center for Water Resources and Spatial Planning (PU SDA TARU) of Central Java Province.

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5.5. Conflicts of Interest

The authors declare no conflict of interest.

6. References

[1] Nurul, N. Y., Wintio, R. C. K., Sutopo, Y., & dan Utomo, K. S. (2017). Redesign of the Spillway of the Cacaban Dam in Tegal Regency. Under Graduates thesis, Universitas Negeri Semarang, Semarang, Indonesia. (In Indonesian).

[2] Anggara, W. W. S., & Sundary, N. (2017). Cacaban Reservoir Volume Change Study with Reservoir Publication Survey. Jurnal Teknik Pengairan: Journal of Water Resources Engineering, 7(2), 310-315. (In Indonesian).

[3] Sutopo Y. (2019). Effect of Spillway Effective Width on Flow Elevation above the Lighthouse and Minimum Cost of Cacaban Dam Renovation. Thesis, Penelitian Unggulan Perguruan Tinggi Unnes, Semarang, Semarang, Indonesia. (In Indonesian).

[4] Hossain, M. M. (2015). Analysis of Flood Routing. Dhaka University Journal of Science, 62(2), 69–73. doi:10.3329/dujs.v62i2.21968.

[5] Ionescu, C. S., & Gogoașe Nistoran, D. E. (2019). Influence of reservoir shape upon the choice of Hydraulic vs. Hydrologic reservoir routing method. E3S Web of Conferences, 85, 1–8. doi:10.1051/e3sconf/20198507001.

[6] Sutapa, I. W. (2019). Study flood routing Mamak Dam and evaluate the River Mamak to convey the flood design, Lombok, Indonesia. IOP Conference Series: Materials Science and Engineering, 673(1), 1–9. doi:10.1088/1757-899X/673/1/012035.

[7] Asmaranto, R., Priyantoro, D., Rini, D. Y., & Aini, A. K. (2020). Safety evaluation of the existing Grawan dam based on hydro-geotechnical behaviour conditions to ensure the availability of water resources. IOP Conference Series: Earth and Environmental Science (Vol. 437, No. 1, p. 012006). IOP Publishing. doi:10.1088/1755-1315/437/1/012006.

[8] Feng, M., & Liu, P. (2014). Spillways Scheduling for Flood Control of Three Gorges Reservoir Using Mixed Integer Linear Programming Model. Mathematical Problems in Engineering (Vol. 2014, pp. 1–9). doi:10.1155/2014/921767.

[9] Karim, I. R., Hassan, Z. F., Abdullah, H. H., & Alwan, I. A. (2021). 2D-Hec-Ras Modeling of Flood Wave Propagation in a Semi-Arid Area Due To Dam Overtopping Failure. Civil Engineering Journal, 7(9), 1501–1514. doi:10.28991/cej-2021-03091739.

[10] NAJAR, M., & GÜL, A. (2022). Investigating the Influence of Dam-Breach Parameters on Dam-Break Connected Flood Hydrograph. Teknik Dergi, 33, 1–24. doi:10.18400/tekderg.796334.

[11] Al Islam, M., & Hasan, H. (2020). Generation of IDF equation from catchment delineation using GIS. Civil Engineering Journal, 6(3), 540–547. doi:10.28991/cej-2020-03091490.

[12] Desa M., M. N., & Rakhecha, P. R. (2007). Probable maximum precipitation for 24-h duration over an equatorial region: Part 2-Johor, Malaysia. Atmospheric Research, 84(1), 84–90. doi:10.1016/j.atmosres.2006.06.005.

[13] Kim, N. W., & Lee, J. (2017). Estimation of time-variant probable maximum precipitation for South Korea. KSCE Journal of Civil Engineering, 21(3), 1031–1038. doi:10.1007/s12205-016-1052-x.

[14] Sibuea, P. R., Agriamah, D. R., Riawan, E., Suwarman, R., & Lubis, A. (2021, November). Estimation of Probable Maximum Precipitation (PMP) and Probable Maximum Flood (PMF) Using GSSHA Model (Case Study Area Upper Citarum Watershed). IOP Conference Series: Earth and Environmental Science (Vol. 893, No. 1, p. 012023). IOP Publishing. doi:10.1088/1755-1315/893/1/012023.

[15] Badan Meteorologi, B. & dan Geofisika, K. (2010). Extreme weather and climate conditions 2010-2011, Jakarta, Indonesia (1st Edition).

[16] Sidibe, M., Dieppois, B., Mahé, G., Paturel, J.-E., Amoussou, E., Anifowose, B., & Lawler, D. (2018). Trend and variability in a new, reconstructed streamflow dataset for West and Central Africa, and climatic interactions, 1950–2005. Journal of Hydrology, 561, 478–493. doi:10.1016/j.jhydrol.2018.04.024.

[17] Chesterton, O. J., Ucuncu, M., & Borman, D. (2019). CFD modelling for dams and reservoirs – best practice workflows, specification and review. Dams and Reservoirs, 29(4), 148–157. doi:10.1680/jdare.19.00032.

[18] Ayu, M., Juwono, P. T., Asmaranto, R., & Milleanisa, K. E. (2021). Dam Break Analysis of Salomekko Dam Using Zhong Xing HY21. IOP Conference Series: Earth and Environmental Science, 930(1), 1–11. doi:10.1088/1755-1315/930/1/012092.

[19] Krisnayanti, D. S., Bolla, M. E., Nasjono, J. K., & Wangge, M. J. M. (2019). The analysis of alpha parameter on Nakayasu Synthetic Unit Hydrograph in Timor Island watersheds. IOP Conference Series: Materials Science and Engineering, 669(1), 1–11. doi:10.1088/1757-899X/669/1/012015.

[20] SNI 8062:2015. (2014). Tata Cara Desain Tubuh Bendungan Tipe Urugan (1st Ed). Standard National Indonesia, Jakarta, Indonesian. Available online: https://adoc.pub/tata-cara-desain-tubuh-bendungan-tipe-urugan.html#google_vignette (accessed on February 2022).

[21] Milleanisa, K. E., Juwono, P. T., Asmaranto, R., & Ayu, M. (2021). Dam Break Analysis of Gembong Dam Using Zhong Xing HY21. IOP Conference Series: Earth and Environmental Science, 930(1), 1–8. doi:10.1088/1755-1315/930/1/012091.

[22] Mediero, L., Jiménez-Álvarez, A., & Garrote, L. (2010). Design flood hydrographs from the relationship between flood peak and volume. Hydrology and Earth System Sciences, 14(12), 2495–2505. doi:10.5194/hess-14-2495-2010.

[23] Volpi, E., Di Lazzaro, M., Bertola, M., Viglione, A., & Fiori, A. (2018). Reservoir Effects on Flood Peak Discharge at the Catchment Scale. Water Resources Research, 54(11), 9623–9636. doi:10.1029/2018WR023866.