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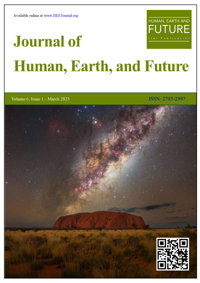

Optimization of Flood Control in the Lake by Gate Opening Simultaneously

Febrian Kusmajaya [1*], Lily M. Limantara [1*], Emma Yuliani [1], Widandi Soetopo [1]

[1] *Department of Water Resources Engineering, Faculty of Engineering, University of Brawijaya, Malang, Indonesia.*

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Abstract

This research aims to optimize the management of Limboto Lake as a flood control measure in Gorontalo Province. Limboto Lake serves as a flood retention basin, receiving inflows from 23 rivers and discharging through a single outlet, the Tapodu Canal, which flows into the Bolango River and ultimately drains into the Bone River (Tomini Bay). Flooding in Gorontalo Province and Gorontalo City often occurs due to the absence of a regulated operation pattern for Limboto Lake. The study employs optimization techniques by simulating various scenarios of Tapodu gate operation under different design flood return periods, considering the canal's capacity of 199.00 m³/s. Given that the lake's storage capacity at an elevation of +7.00 m is 172.465 million m³, the embankment remains structurally safe for floods with a return period of up to 1,000 years. However, in the event of a Probable Maximum Flood (PMF), the embankment is expected to experience overtopping by 0.28 cm. Optimization results indicate that, without gate regulation, the outflow and maximum lake elevation reach 369.74 m³/s at +6.09 m for a 1,000-year flood and 408.80 m³/s at +6.84 m for a PMF event. In contrast, with gate regulation, these values are significantly reduced to 110.50 m³/s at +6.58 m for a 1,000-year flood and 142.00 m³/s at +7.28 m for a PMF event. Given that the Tapodu Canal's capacity is 199.00 m³/s and the lake's elevation is +7.00 m, uncontrolled outflows could lead to an overflow of the Tapodu Canal by 0.28 cm.

Keywords: Flood; Gorontalo; Lake; Limboto; Optimization.

1. Introduction

The management of a lake or dam's storage capacity depends on reservoir operation regulations, which determine both minimum and maximum water levels. During flood seasons, the reservoir's water level must not be too high, as it could trigger severe flooding, nor too low, as it would impact water availability downstream [1]. One crucial parameter in optimizing reservoir storage is the Flood Limit Water Level (FLWL). A study by Liu et al. [2] found that optimizing storage utilization for both flood control and hydropower generation in four cascade dams—across 20 trials—showed that dynamic programming is more suitable for normal-year conditions than for wet-year conditions. However, in dry-year conditions, dynamic programming could not be applied effectively.

Flood control methods involve not only structural techniques but also non-structural approaches, such as optimizing reservoir operation patterns [3, 4]. Numerous flood control models have been developed with varying objectives [5], most of which focus on minimizing downstream flooding and reducing the reservoir's maximum water level. Zhu et al. [6] conducted research on optimizing reservoir operation patterns for both flood control and other reservoir functions. Their study applied Dynamic Programming (DP) and the DP-POA (Progressive Optimality Algorithm) method. The

* Corresponding author: lilymont@ub.ac.id; iyanjaya34@gmail.com

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DP-POA approach was particularly effective in minimizing peak floods downstream by lowering the reservoir's water level before the maximum flood peak occurred. Similarly, Wang et al. [7] utilized dynamic programming to assess the effectiveness of the Dongjiang Reservoir in reducing flood peaks at an hourly interval. Their results demonstrated that optimization through dynamic programming reduced flood duration by 106 seconds (a 65% decrease) for the first flood event and by 37 seconds (a 59% decrease) for the second flood event.

Reservoir optimization is not limited to dynamic programming (DP); other methods, such as Differential Evolution (DE), have also been employed. Phuong et al. [8] compared DP and DE, finding that their results were closely aligned. However, their effectiveness varied based on inflow levels. When inflow was low, DE yielded higher electricity production, whereas in high-inflow conditions, DP produced more electricity. Additionally, Jiang et al. [9] applied dynamic programming to optimize the multi-dimensional operation of cascade reservoirs used for hydropower generation. Their findings indicated that water level reduction was most effective in the first release stage when the reservoir followed a two-stage water release strategy during the non-flood end-season. Moreover, variations in downstream reservoir water levels during flood seasons—except in the initial and final stages—enhanced electricity generation compared to conventional methods.

Liu et al. [10] conducted research on flood management in downstream areas, proposing a methodology that divides flood control into three scenarios, one of which focuses on optimizing flood diversion locations and implementing gate control strategies. This approach was successfully applied in the Huayanghe Detention Basin (HDB) along the Yangtze River. Their findings indicated that controlled water gate operations could significantly reduce flood damage within the detention basin, particularly for relatively minor flood events. Various models have been developed for flood control operations, each serving different objectives. Wang et al. [11] explored the complexities of water resource management in lakes that are directly connected to rivers. Their study incorporated uncertainty analysis to develop a multi-objective optimization framework, maximizing the storage capacity of lakes while balancing other hydrological factors.

Flood control optimization has also been explored through simulation-based methodologies, as demonstrated by Ding et al. [12]. Their study employed a nonlinear numerical optimization approach to simulate flow behavior, aiming to determine the optimal flood detention strategy for reducing water levels in a channel network within a watershed. The research assessed the effectiveness of single and multiple water gates in managing flood detention. Their findings showed that installing multiple water gates offers a cost-effective solution for evenly distributing floodwater across a watershed, preventing rapid river flow acceleration—a common issue associated with single-gate control systems. Numerical optimization results demonstrated that the developed model efficiently identified optimal solutions across various control conditions.

Similarly, Lianqing et al. [13] investigated flood management optimization within lake systems, simulating one- and two-dimensional flood scenarios in the Dongting Lake system. Their study focused on controlling water levels, determining optimal flood detention locations, and refining gate regulation strategies. The results indicated that precise gate adjustments could significantly reduce high-water level risks. For specific flood events, leveraging the lake system's natural storage capacity and implementing a multi-tiered flood control approach proved to be highly effective in enhancing overall flood management.

Flooding has become a recurring problem in Gorontalo Province in recent years. Both urban and rural areas near rivers frequently experience inundation due to river overflow and excessive rainfall that cannot be absorbed or effectively drained. These floods have caused significant damage to public infrastructure, the environment, and residential areas, resulting in both human casualties and substantial property losses [14].

Limboto Lake, located in Gorontalo Province, is part of a lagoon system with its outflow connected to the Bolango and Bone Rivers. The lake receives water from 23 tributaries that flow in from the northern, western, and southern watersheds, covering a total area of 4,419.72 hectares, as defined by the Ministry of Public Works and Housing's Decision No. 350/KPTS/M/2023 regarding the boundary of the Limboto-Bolango-Bone river region. The recurrent flooding in Gorontalo Province, particularly in Gorontalo Regency and Gorontalo City, is largely attributed to the lack of an operational management plan for Limboto Lake. The lake's only outlet, the Tapodu River, eventually drains into the Bolango River, which then merges with the Bone River before reaching Tomini Bay.

The effectiveness of lake or reservoir storage for flood control depends on several factors, including inflow levels, the capacity of downstream rivers [15], and overall storage capacity [16]. Optimizing storage operations is a critical and efficient strategy to enhance flood control capacity [5]. Two key approaches for managing lake storage are simulation and optimization. While optimization focuses on selecting the best option among multiple alternatives, simulation aims to model system behavior for a better understanding of its dynamics [17]. To implement an effective flood management strategy, it is essential to estimate the available flood resources. By analyzing historical inflow and outflow data over a given period, the annual flood season and the required water release volume can be determined [18].

This study aims to optimize the management of Limboto Lake for flood control in Gorontalo Province by coordinating gate operations using simulation techniques. With comprehensive data availability, the optimization results will provide an effective lake management strategy to mitigate flooding.

2. Material and Methods

2.1. Existing Condition

2.1.1. Watershed (DTA)

Limboto Lake is located in Gorontalo Regency, Gorontalo Province, Indonesia, within the Limboto–Bolango–Bone (LBB) River Region. It is part of the Bolango-Limboto watershed, which covers an area of approximately 1,082.2 km². A spatial analysis using GIS software was conducted to determine the area of each sub-watershed within the Limboto Lake watershed. The watershed is divided into 79 sub-watersheds, as detailed in Table 1.

Table 1. Division of Sub-watershed in Limboto Lake

No.	Sub Basin	Area (km^2)	No.	Sub Basin	Area (km^2)	No.	Sub Basin	Area (km^2)
1	Sub DAS 1	26.72	21	Sub DAS 28	17.61	41	Sub DAS 46	4.52
2	Sub DAS 10	6.42	22	Sub DAS 29	6.57	42	Sub DAS 47	15.99
3	Sub DAS 11	47.8	23	Sub DAS 3	15.24	43	Sub DAS 48	1.3
4	Sub DAS 12	15.24	24	Sub DAS 30	7.27	44	Sub DAS 49	4.01
5	Sub DAS 13	10,16	25	Sub DAS 31	21.71	45	Sub DAS 5	17.48
6	Sub DAS 14	6.48	26	Sub DAS 32	5.93	46	Sub DAS 50	9.96
7	Sub DAS 15	19.09	27	Sub DAS 33	21.59	47	Sub DAS 51	7.5
8	Sub DAS 16	9.08	28	Sub DAS 34	5.59	48	Sub DAS 52	8.01
9	Sub DAS 17	22.23	29	Sub DAS 35	10.54	49	Sub DAS 53	4.81
No.	**Sub Basin**	**Area (km^2)**	**No.**	**Sub Basin**	**Area (km^2)**	**No.**	**Sub Basin**	**Area (km^2)**
10	Sub DAS 18	11.24	30	Sub DAS 36	6.64	50	Sub DAS 54	0
11	Sub DAS 19	13.47	31	Sub DAS 37	10.87	51	Sub DAS 55	1.45
12	Sub DAS 2	29.2	32	Sub DAS 38	19.02	52	Sub DAS 56	1.44
13	Sub DAS 20	7.36	33	Sub DAS 39	3.32	53	Sub DAS 57	0.67
14	Sub DAS 21	7.79	34	Sub DAS 4	10.83	54	Sub DAS 58	5.42
15	Sub DAS 22	6.64	35	Sub DAS 40	4.23	55	Sub DAS 59	16.75
16	Sub DAS 23	6.21	36	Sub DAS 41	1.82	56	Sub DAS 6	7.29
17	Sub DAS 24	11.33	37	Sub DAS 42	9.49	57	Sub DAS 60	5.05
18	Sub DAS 25	22.69	38	Sub DAS 43	12.82	58	Sub DAS 61	17.08
19	Sub DAS 26	22.95	39	Sub DAS 44	34.06	59	Sub DAS 62	3.17
20	Sub DAS 27	10.37	40	Sub DAS 45	0.41	60	Sub DAS 63	0.29
No.	**Sub Basin**	**Area (km^2)**	**No.**	**Sub Basin**	**Area (km^2)**			
61	Sub DAS 64	0.36	72	Sub DAS 74	0.77			
62	Sub DAS 65	0.32	73	Sub DAS 75	1.14			
63	Sub DAS 66	11.14	74	Sub DAS 76	1.53			
64	Sub DAS 67	1.68	75	Sub DAS 77	6.81			
65	Sub DAS 68	4.69	76	Sub DAS 78	3.8			
66	Sub DAS 69	0.05	77	Sub DAS 79	0.37			
67	Sub DAS 7	12.09	78	Sub DAS 8	10.69			
68	Sub DAS 70	1.19	79	Sub DAS 9	12.94			
69	Sub DAS 71	4.33						
70	Sub DAS 72	0.01						
71	Sub DAS 73	0.13						

2.1.2. Land Cover

From the land cover map above, it can be known the composition of land cover in Limboto watershed as presented in Table 2.

Table 2.Composition of Land Cover in Limboto Lake Watershed

Land Use Class	Area (km^2)	%
Secondary Dry land Forest	73.39	9.9%
Thicket	68.94	9.3%
Open Land	0,22916667	0.7%
Mixed Dry Land Agriculture	263.21.00	35.6%
Plantation	03.00	0.4%
Housing	24.55.00	3.3%
Swamp Thicket	0.27708333	0.8%
Dry land farming	220.52.00	29.9%
Field Rice	73.05.00	9.9%
Airport / Harbour	0.05625	0.1%

2.1.3. Lake Storage

In almost every rainy season, the Limboto Lake is flooding, it is due to the lake storage power decreasing. The inundated area in the rainy season is in surrounding lake in downstream area, Bivongga River and Tapodu River and along Alo Pohu area. Figure 1 presents the storage capacity of Limboto Lake in 2020 and Table 3 presents the elevation and inundation in Limboto Lake.

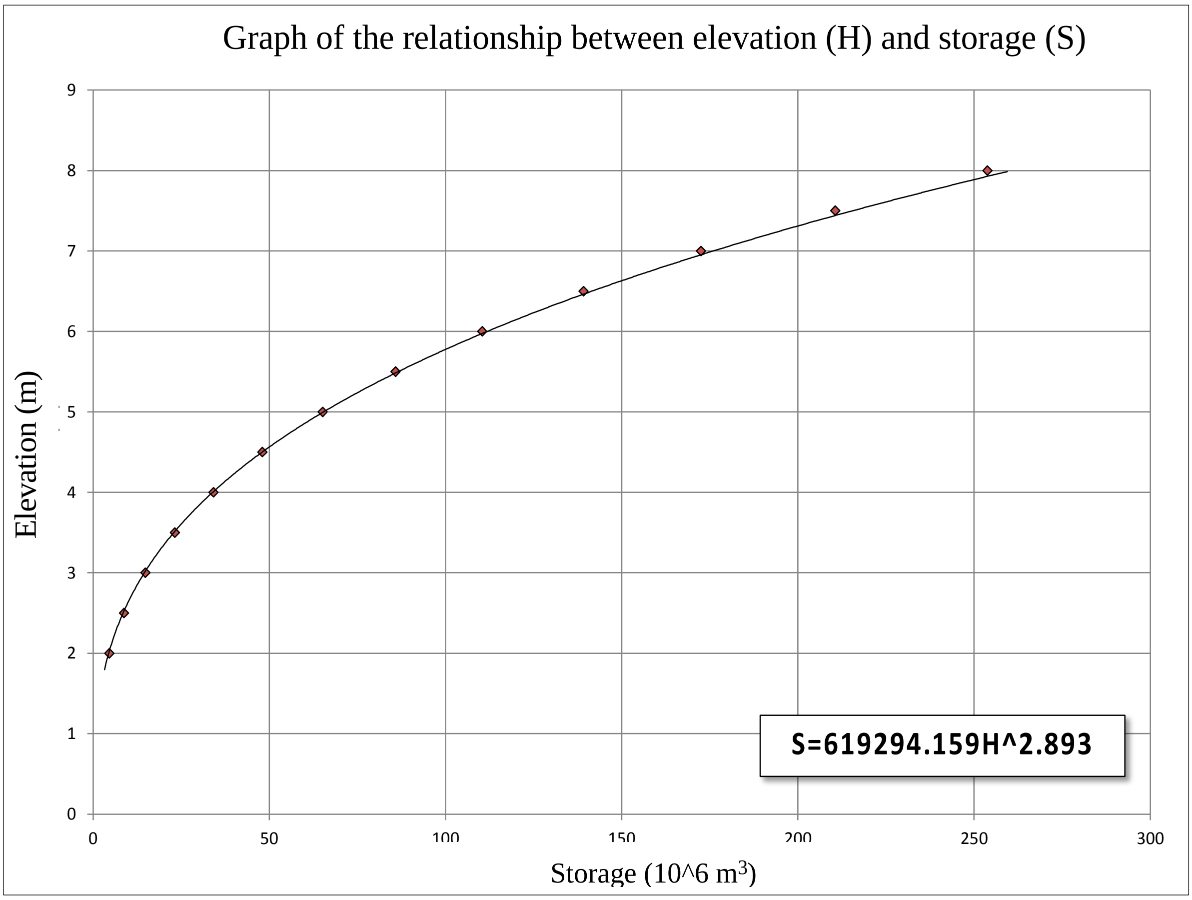

Figure 1. Graph of the Relationship between Elevation (H) and Storage (S)

Table 3. Elevation and Inundation in Limboto Lake

No.	Elevation H (m)	ΔH (m)	Inundation Area		Volume		Category
			m^2	ha	10^3m^3	Million m^3	
1	1.50	0	2,181.395	218.139457	-	-	
2	2.00	0.5	6,788.289	678.828871	4,599.96	4.600	Low
3	2.50	1	13,833.488	1,383.34875	8,772.17	8.772	
4	3.00	1.5	20,636.400	2,063.63998	14,865.25	14.865	
5	3.50	2	24,892.297	2,489.22972	23,219.04	23.219	Normal
6	4.00	2.5	27,411.855	2,741.18547	34,167.29	34.167	
7	4.50	3	31,592.104	3,159.21037	48,038.66	48.039	
8	5.00	3.5	35,832.039	3,583.20388	65,157.41	65.157	High
9	5.50	4	39,940.392	3,994.03924	85,843.95	85.844	
10	6.00	4.5	43,039.509	4,303.95085	110,415.27	110.415	
11	6.50	5	45,892.320	4,589.23198	139,185.26	139.185	
12	7.00	5.5	48,821.770	4,882.17703	172,465.04	172.465	
13	7.50	6	51,105.973	5,110.59725	210,563,.16	210.563	
14	8.00	6.5	52,874.500	5,287.45001	253,785.82	253.786	

Source: River Basin Organization of Sulawesi II Gorontalo, 2018.

The bathymetry measurement result in 2020 showed the storage capacity of Limboto Lake now in normal condition that is about 34,167,290 m^3 and in flood condition is about 85,843,950 m^3.

2.2. Design Rainfall

The probability of rainfall frequency [19] is used to analyze design rainfall. Several methods are available and are selected based on data suitability, including: a) Generalized Extreme Value (GEV); b) Iwai-Kadoya; c) Log Pearson III; d) Gumbel. Hydro-gnomon software is then used to analyze the design rainfall.

After calculating design rainfall using these four methods, the results are tested for distribution suitability both vertically using the Chi-Square test and horizontally using the Smirnov-Kolmogorov test. As an example, the analysis results from Hydro-gnomon for the GPM2 station are presented.

Based on the Chi-Square and Smirnov-Kolmogorov test results, the distribution with the highest reliability is determined to be the Normal distribution. The same process is applied to other rainfall stations. Table 4 presents the rainfall stations within the Limboto Lake watershed.

Table 4. Design Rainfall in Limboto Lake Watershed

No,	Rainfall Station	Distribution	1.01	2	5	10	20	25	50	100	1000
1	Limboto Datahu	Log Normal	28.61	72.83	102.08	121.77	140.87	146.98	165.98	185.15	251.52
2	Limboto Tabongo	Normal	31.03	73.40	88.70	96.70	103.31	105.24	110.75	115.70	129.60
3	Limboto Pilolalenga	Log Normal	40.34	71.25	87.49	97.41	106.44	109.23	117.61	125.70	151.47
4	GPM 2	Normal	14.25	105.27	138.15	155.34	169.53	173.66	185.50	196.15	225.99
5	GPM 3	Log Normal	31.22	89.30	130.54	159.19	187.54	196.71	225.52	255.03	359.95
6	GPM 6	Log Normal	28.25	89.12	134.95	167.64	200.53	211.27	245.32	280.61	408.95
7	GPM 7	Normal	2.56	88.68	119.79	136.05	149.48	153.39	164.59	174.67	202.91

2.3. Design Flood

There is not available flood observed discharge. Therefore, the unit hydrograph analysis is used for analysing the design flood. However, the unit hydrograph approach consists of direct-run-off that is produced by more evenly rainfall in all of watershed with fixed intensity in time unit (Figure 2).

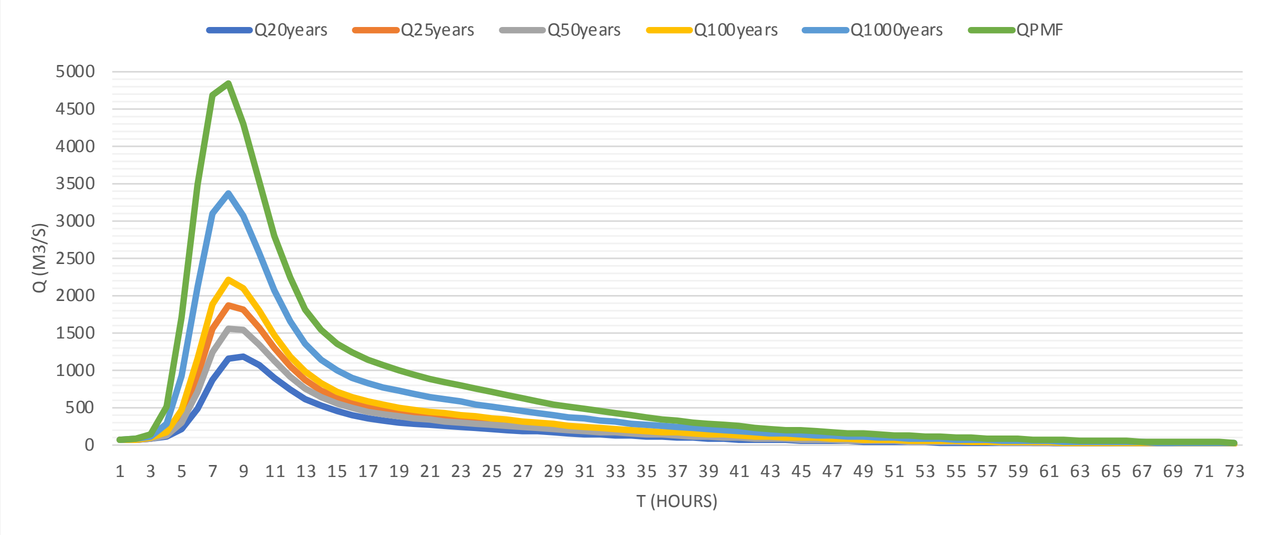

Figure 2. Design Flood Hydrograph of SCS-CN

The result of flood modelling by using SCS IUH – CN is as follows (Table 5):

Table 5. Result of Flood Modelling

Time	$Q_{20\ years}$	$Q_{25\ years}$	$Q_{50\ years}$	$Q_{100\ years}$	$Q_{1000\ years}$	Q_{PMF}
00.00	75.3	75.3	75.3	75.3	75,3	75.3
01.00	77.5	78.5	78.1	79	80,5	82.4
02.00	86.5	91.8	89.3	94.9	110,5	142
03.00	115	150.5	132.6	172.7	283,7	511.2
04.00	214.7	375.9	294.4	474.8	929	1,717.1
05.00	481.7	922	708	1,161.5	2,111.5	3,484.3
06.00	875	1,557.8	1,241.3	1,890	3,094.3	4,679.5
07.00	1154.7	1,876.7	1,553.2	2,205.8	3,364	4,839.4
08.00	1189.7	1,812.9	1,536.5	2,092	3,069.1	4,293.4
09.00	1064.7	1,571.8	1,348.1	1,796.1	2,568.2	3,524.2
10.00	898.7	1,294.6	1,121.8	1,468.4	2,063.5	2,799.6
11.00	741	1,053	915.6	1,191.1	1,660.5	2,234.1
12.00	615.5	869.8	756.7	980.5	1,355.8	1,818.4
13.00	523.8	735.7	639.7	826.3	1,144.9	1,544.1
14.00	456.1	640.4	556.4	717.4	1,000	1,358.5
15.00	404.5	571.1	495.1	640.7	903.5	1,239.9
16.00	362.8	517.9	447.5	584.5	831.6	1,145.6
17.00	329.8	475.9	410	539.8	773	1,069.8
18.00	303.4	442.3	379.8	503.2	722.6	1,000.1
19.00	284.3	416.1	356.9	473.3	679.5	941.6
20.00	267.9	393.4	336.9	447.4	641.3	890.6
21.00	255.1	376.4	321.6	427.1	610.6	848.2
22.00	243.4	359.5	307.3	406.4	580.5	804.1
23.00	231.7	341.7	292.6	384.9	550.3	758.8
00.00	219	322.2	276.7	361.9	518.2	712.4
01.00	206.3	302.7	260.6	339.8	486.8	668.2
02.00	193.8	283.7	244.6	318.6	456	625.7
03.00	181.8	266.1	229.4	298.6	427.6	586.5

04.00	170.6	249.4	215.1	280	400.5	549.2
05.00	160	233.9	201.7	262.5	375.4	514.7
06.00	150.1	219.3	189.2	246.1	351.8	482.1
07.00	140.8	205.6	177.5	230.7	329.7	451.7
08.00	132.2	192.8	166.4	216.3	309	423.2
09.00	124	180.9	156.2	202.9	289.7	396.6
10.00	116.4	169.7	146.5	190.3	271.6	371.7
11.00	109.3	159.2	137.5	178.5	254.6	348.4
12.00	102.6	149.4	129.1	167.5	238.8	326.6
13.00	96.4	140.2	121.2	157.2	224	306.3
14.00	90.6	131.6	113.8	147.5	210.1	287.2
15.00	85.1	123.6	106.9	138.5	197.1	269.3
16.00	80	116.1	100.4	130	184.9	252.6
17.00	75.1	109	94.3	122.1	173.6	236.9
18.00	70.6	102.4	88.6	114.6	162.9	222.2
19.00	66.4	96.2	83.3	107.7	152.9	208.5
20.00	62.4	90.4	78.2	101.2	143.5	195.7
21.00	58.7	84.9	73.5	95	134.8	183.6
22.00	55.2	79.8	69.1	89.3	126.6	172.4
23.00	51.9	75	65	84	118.9	161.8
00.00	48.9	70.6	61.1	78.9	111.7	151.9
01.00	46	66.4	57.5	74.2	104.9	142.7
02.00	43.3	62.4	54.1	69.8	98.6	134
03.00	40.8	58.7	50.9	65.6	92.7	125.9
04.00	38.4	55.2	47.9	61.7	87.2	118.3
05.00	36.2	52	45.1	58.1	82	111.1
06.00	34.1	48.9	42.5	54.7	77.1	104.5
07.00	32.1	46.1	40.1	51.5	72.5	98.2
08.00	30.3	43.4	37.7	48.5	68.2	92.4
09.00	28.6	40.9	35.6	45.6	64.2	86.9
10.00	27	38.6	33.5	43	60.4	81.7
11.00	25.4	36.3	31.6	40.5	56.9	76.9
12.00	24	34.3	29.8	38.2	53.6	72.4
13.00	22.7	32.3	28.1	36	50.5	68.1
14.00	21.4	30.5	26.6	34	47.6	64.2
15.00	20.2	28.8	25.1	32	44.8	60.4
16.00	19.1	27.2	23.7	30.2	42.3	56.9
17.00	18.1	25.6	22.4	28.5	39.9	53.7
18.00	17.1	24.2	21.1	27	37.6	50.6
19.00	16.2	22.9	20	25.5	35.5	47.7
20.00	15.3	21.6	18.9	24.1	33.5	45
21.00	14.5	20.5	17.9	22.8	31.7	42.5
22.00	13.8	19.4	16.9	21.5	29.9	40.1
23.00	13.1	18.3	16	20.4	28.3	37.9
00.00	12.4	17.4	15.2	19.3	26.7	35.8

2.4. Outlet of Limboto Lake

The outlet of Limboto Lake features two outflow regulation systems: the Tapodu gate and a spillway. These systems are used to manage and control the water level elevation around Limboto Lake. Figure 3 illustrates the layout of the Tapodu gate, Figure 4 shows its longitudinal section, and Figure 5 presents its cross-sectional view.

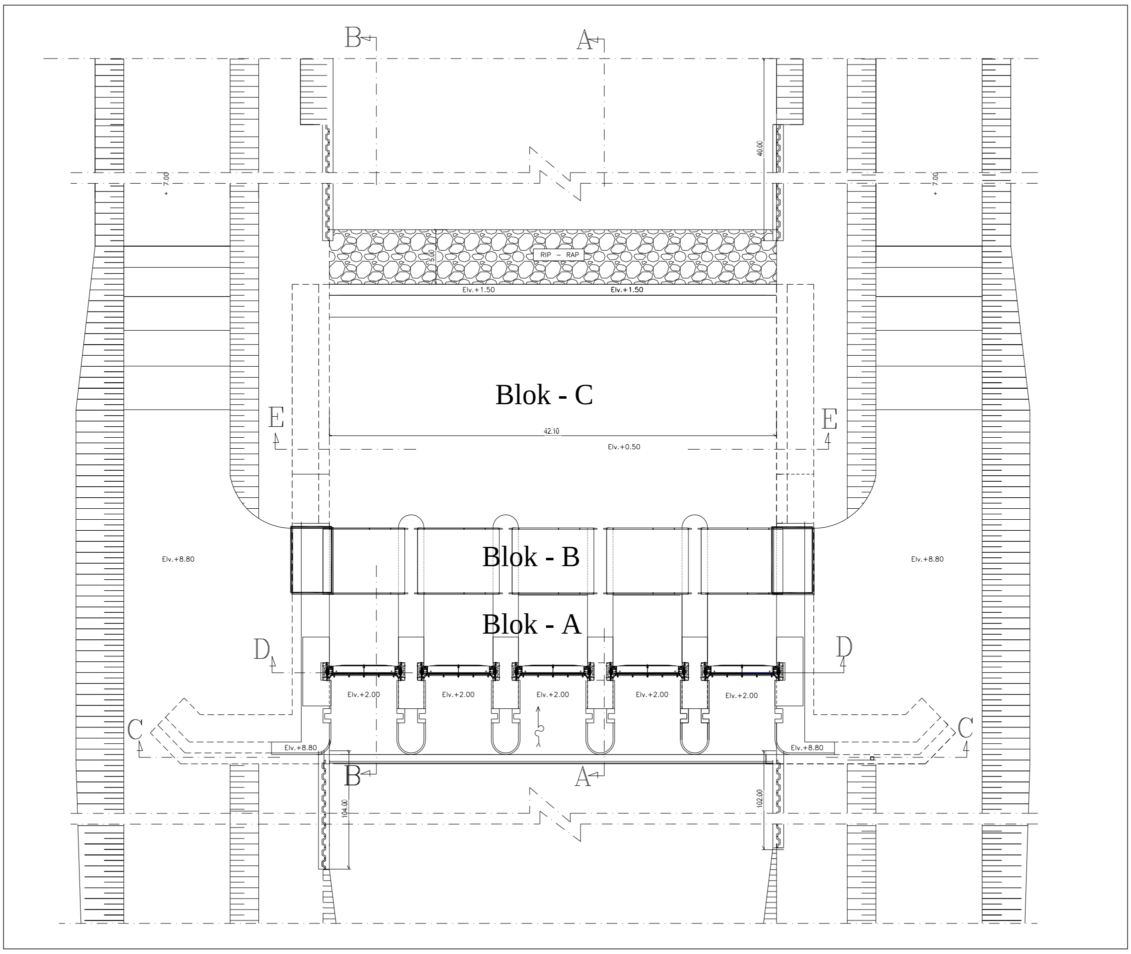

Figure 3. Lay Out of Tapodu Gate

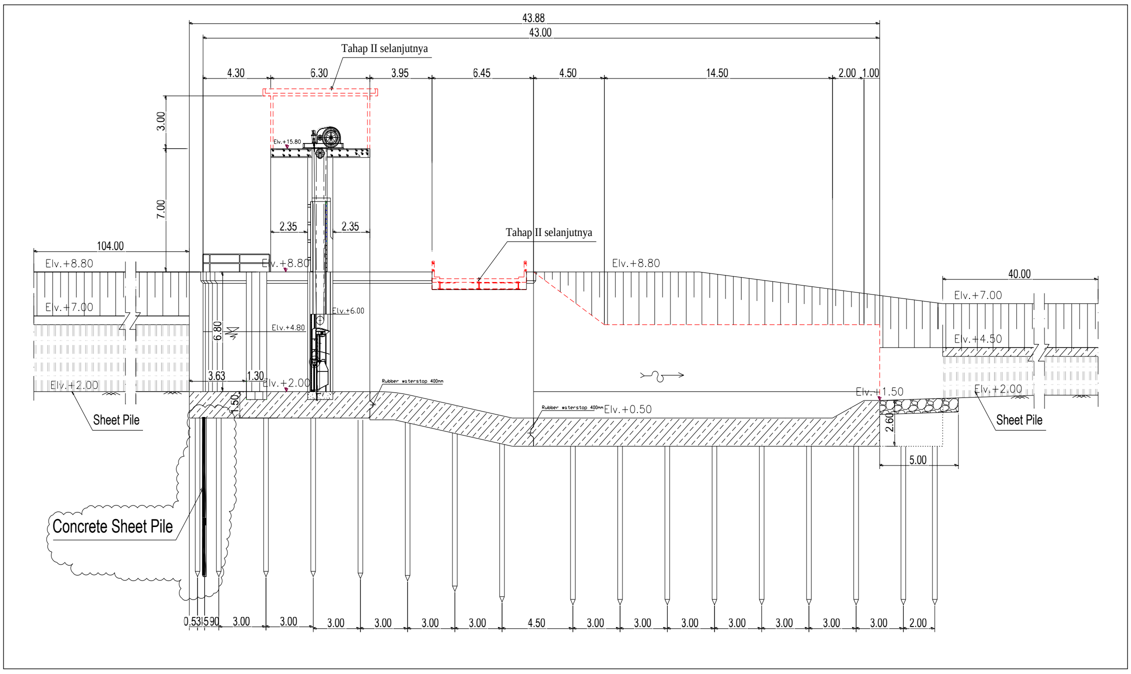

Figure 4. Long Section of Tapodu Gate

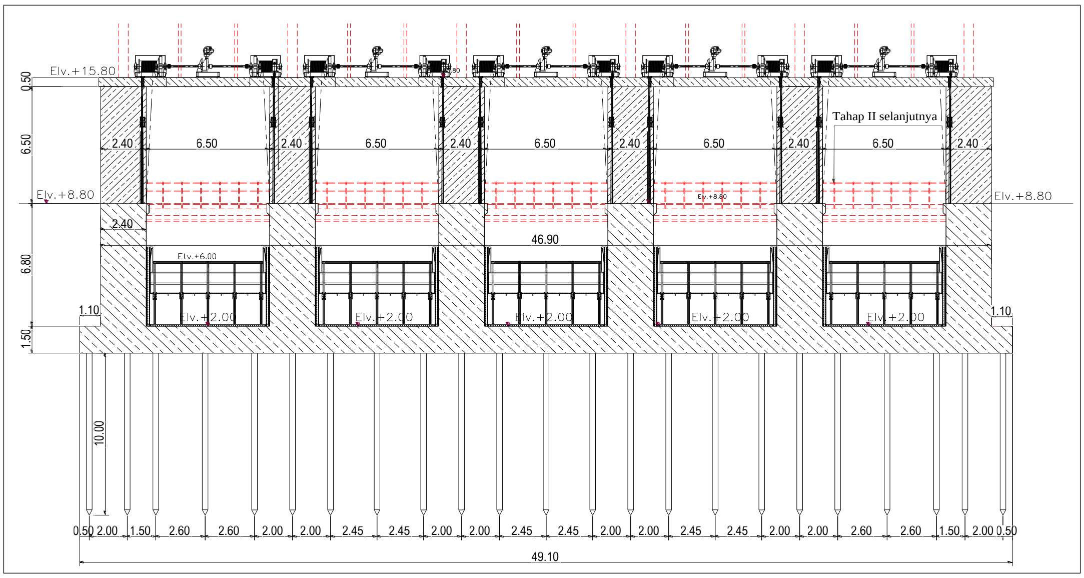

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Figure 5. Cross Section of Tapodu Gate

For the condition of the study now is carried out the optimization of outflow in Tapodu gate opening with the technical data as follows:

1. Width of gate = 6.50 m
2. Number of gates = 5.00 Units
3. Elevation of bed gate = + 2.00 m
4. Maximum gate opening = 4.00 m / Elevation + 4.00 m

2.5. Tapodu Canal of Limboto Lake

Outflow from Tapodu gate outlet is directed through Tapodu canal with the length total is 1,900 m and the estuary will be in Bolango river. The capacity of Tapodu canal for being flow the maximal discharge is 199.00 m^3/s. Figure 6 presents the cross section of Tapodu canal and Figure 7 presents the trace of Tapodu canal.

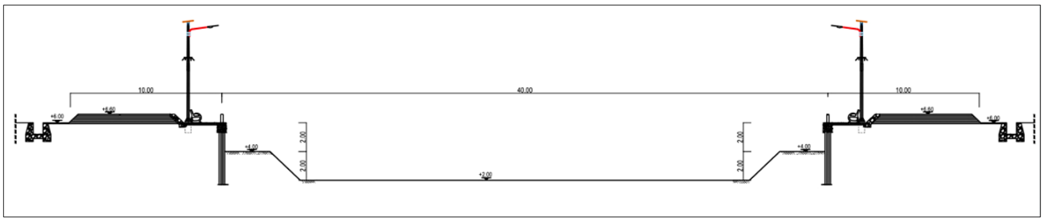

Figure 6. Cross Section of Tapodu Canal

Figure 7. Trace Tapodu Canal

2.6. Optimization method

System analysis is a method for studying and analyzing various aspects of a system. Analysis of water resource systems, especially in flood management, aims to modify flow behavior by utilizing natural conditions and existing boundaries. Figure 8 presents the research flowchart.

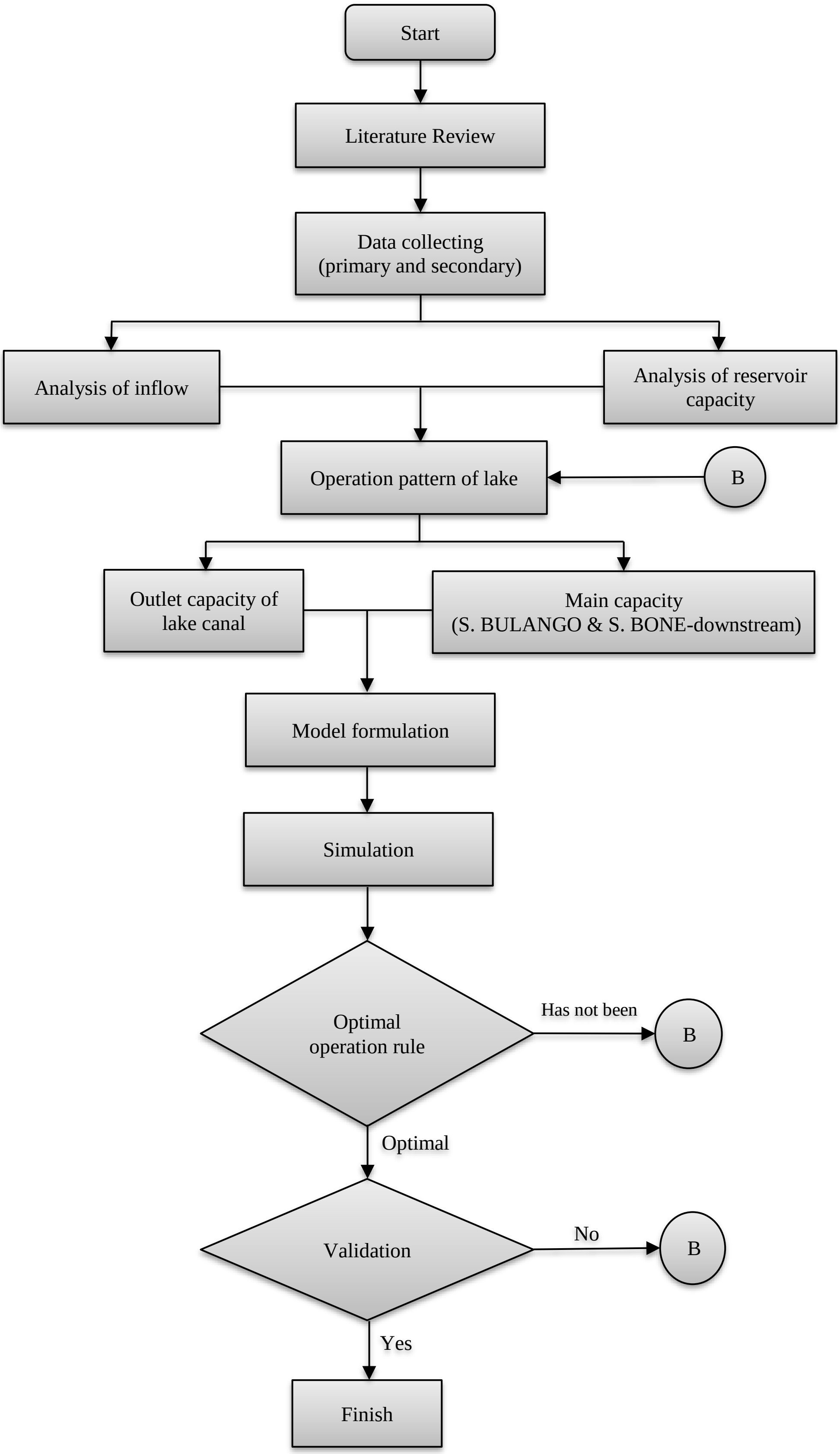

Figure 8. Research Flowchart

By using the system analysis method, it is hoped that water resource management, especially in flood control, can be relied upon optimally and managed in accordance with existing facilities and infrastructure. According to Warren A. Hall and John A. Dracup, a system is a collection of functional components that is interconnected in various ways, where the system requires input and produces output.

The simulation method that is used in writing this journal is by using the law of water balance. The water balance in the reservoir/storage is as follows:

$$S_{t+1} = S_t + I - O \tag{1}$$

with S_{t+} is end period-t storage, S_t is starting period storage, I is inflow total to reservoir, and O is outflow total from reservoir.

Basically, the reservoir balance equation is a reservoir/storage operation which is an inventory for a certain period regarding the amount of water coming in and out and its effect on the reservoir.

3. Results and Discussion

3.1. The Relation between Inflow of Tapodu Canal and Storage of Limboto Lake

Input Data:

- Coefficient of gate discharge = 4.416696 m
- Time of simulation= 3,600 seconds
- A = 619,294.159 m (storage equation)
- p = 2.893 m (storage equation)
- Gate opening= 1.40 m
- Elevation of lake-bed= 2.00 m

By using the continuity equation ($I - Q = \frac{dS}{dT}$) [16], there is obtained the curve of lake storage as follows: (Figure 9)

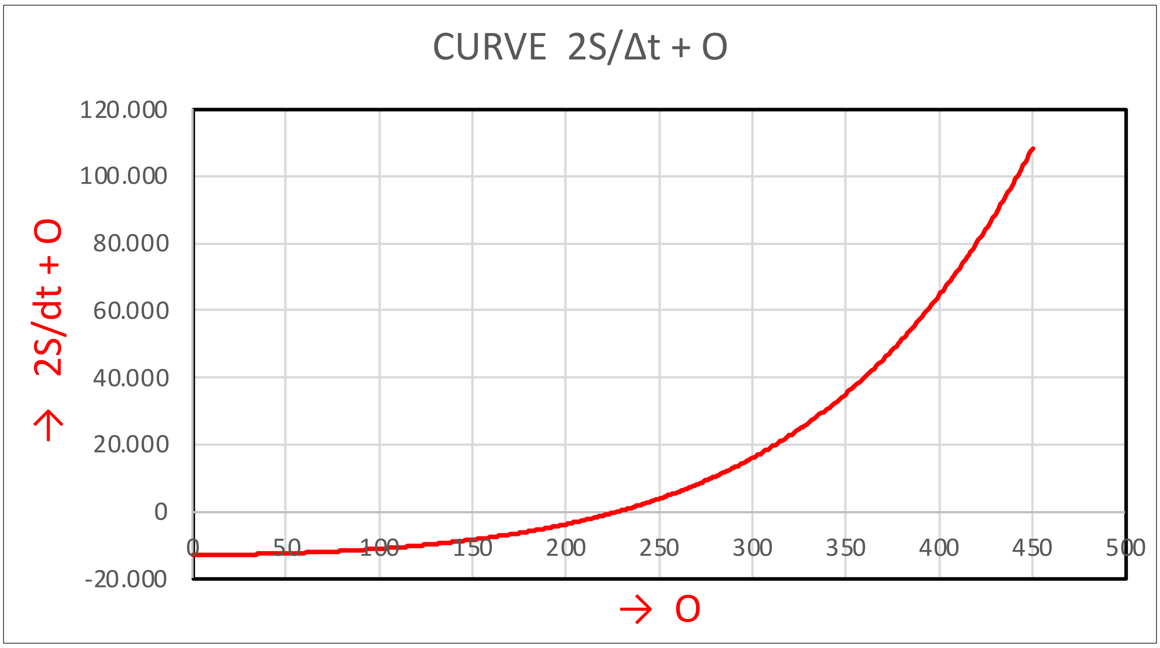

Figure 9. Relation Curve of Inflow and Storage

3.2. Relation between Outflow of Tapodu Gate and Storage of Limboto Lake

Limboto Lake, serving as a storage reservoir for the flow of 23 rivers (affluents), helps reduce the outflow into the Bolango River, which eventually drains into Bone River and then flows toward Tomini Bay. The analysis is conducted under two scenarios: one without the regulator (existing condition) and one with the Tapodu gate, which directs water through the Tapodu Canal with a maximum capacity of 199 m³/s. The results of the analysis are presented in Figures 10 and 11, as well as in Tables 6 and 7.

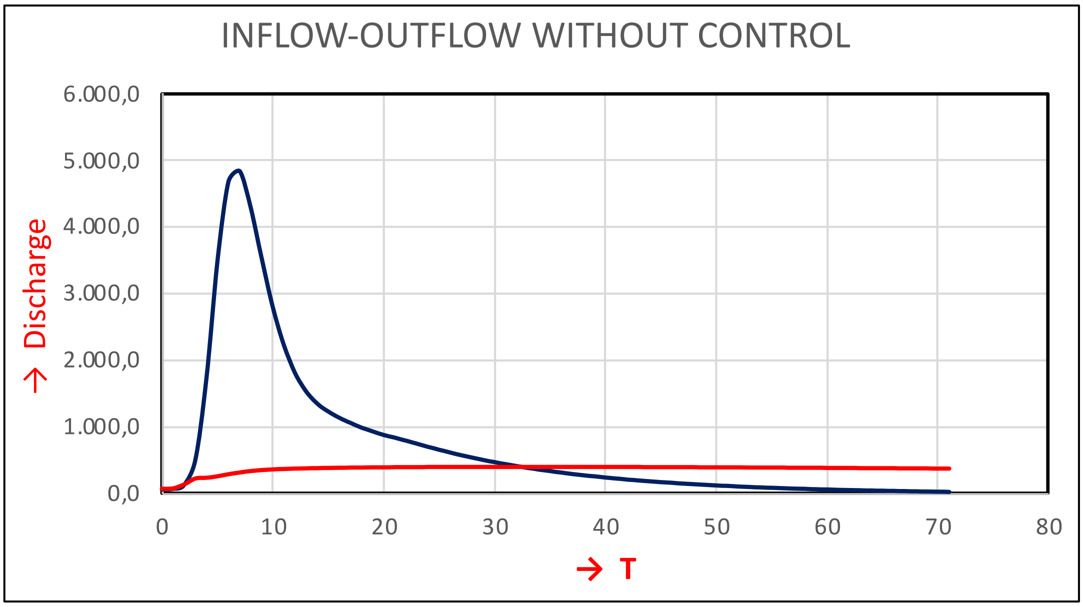

Figure 10. Relation Curve between Outflow and Storage (without gate)

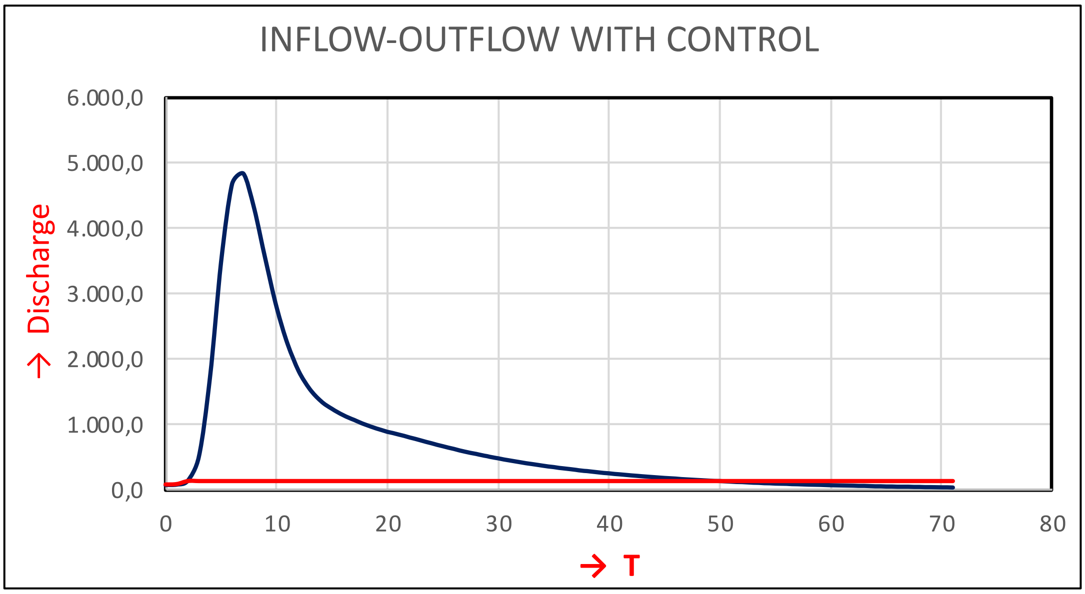

Figure 11. Relation Curve between Outflow and Storage (with gate)

Table 6. Analysis Result of the Relation between Outflow and Storage Due to the Outflow Condition of Q_{PMF}

		Without Control		With Control	
T	**I**	**O**	**Sb**	**O**	**Sb**
[hours]	**[m³/s]**	**[m³/s]**	**[m³/s].[hours]**	**[m³/s]**	**[m³/s].[hours]**
0	75.3	75.30	0.00	75.30	0.00
1	82,4	82.40	0.00	82.40	0.00
2	142,0	142.00	0.00	142.00	0.00
3	511,2	230.91	140.14	142.00	184.60
4	1,717.1	241.43	1,018.12	142.00	1,156.75
5	3,484.3	265.27	3,365.47	142.00	3,615.45
6	4,679.5	295.25	7,167.11	142.00	7,555.35
7	4,839.4	322.34	11,617.77	142.00	12,172.80
8	4,293.4	343.21	15,851,39	142.00	16,597.20

		Without Control		With Control	
T	**I**	**O**	**Sb**	**O**	**Sb**
[hours]	**[m³/s]**	**[m³/s]**	**[m³/s].[hours]**	**[m³/s]**	**[m³/s].[hours]**
9	3,524.2	358.23	19,409.47	142.00	20,364.00
10	2,799,6	368.83	22,207.84	142.00	23,383.90
11	2,234.1	376.36	24,352.09	142.00	25,758.75
12	1,818,4	381.83	25,999.25	142.00	27,643.00
13	1,544.1	385.98	27,296.59	142.00	29,182.25
14	1,358.5	389.27	28,360.27	142.00	30,491.55
15	1,239.9	392.02	29,268.82	142.00	31,648.75
16	1,145.6	394.38	30,068.37	142.00	32,699.50
17	1,069.8	396.45	30,780.65	142.00	33,665.20
18	1,000.1	398.28	31,418.23	142.00	34,558.15
19	941.6	399.89	31,989.99	142.00	35,387.00
20	890.6	401.33	32,505.49	142.00	36,161.10
21	848.2	402.62	32,972.91	142.00	36,888.50
22	804.1	403.77	33,395.87	142.00	37,572.65
23	758.8	404.79	33,773.04	142.00	38,212.10
24	712.4	405.68	34,103.41	142.00	38,805.70
25	668.2	406.43	34,387.66	142.00	39,354.00
26	625.7	407.07	34,627.85	142.00	39,858.95
27	586.5	407.59	34,826.62	142.00	40,323.05
28	549.2	408.02	34,986.67	142.00	40,748.90
29	514.7	408.34	35,110.44	142.00	41,138.85
30	482.1	408.57	35,200.38	142.00	41,495.25
31	451.7	408.73	35,258,63	142.00	41,820.15
32	423.2	408.80	35,287.32	142.00	42,115.60
33	396.6	408.80	---	142.00	42,383.50
34	371.7	408.74	---	142.00	42,625.65
35	348.4	408.61	---	142.00	42,843.70
36	326.6	408.43	---	142.00	43,039.20
37	306.3	408.19	---	142.00	43,213.65
38	287.2	407.90	---	142.00	43,368.40
39	269.3	407.55	---	142.00	43,504.65
40	252.6	407.17	---	142.00	43,623.60
41	236.9	406.74	---	142.00	43,726.35
42	222.2	406.27	---	142.00	43,813.90
43	208.5	405.76	---	142.00	43,887.25
44	195.7	405.22	---	142.00	43,947.35
45	183.6	404.64	---	142.00	43,995.00
46	172.4	404.03	---	142.00	44,031.00
47	161.8	403.38	---	142.00	44,056.10
48	151.9	402.71	---	142.00	44,070.95
49	142.7	402.01	---	142.00	44,076.25
50	134.0	401.28	---	142.00	---
51	125.9	400.53	---	142.00	---
52	118.3	399.75	---	142.00	---
53	111.1	398.95	---	142.00	---
54	104.5	398.13	---	142.00	---
55	98.2	397.28	---	142.00	---
56	92.4	396.42	---	142.00	---
57	86.9	395.53	---	142.00	---
58	81.7	394.62	---	142.00	---
59	76.9	393.70	---	142.00	---
60	72.4	392.76	---	142.00	---
61	68.1	391.80	---	142.00	---
62	64.2	390.82	---	142.00	---

Table 7. Recapitulation of the Relation between Outflow and Storage

Used Inflow Hydrograph Series		Unit	$Q_{20\ years}$	$Q_{25\ years}$	$Q_{50\ years}$	$Q_{100\ years}$	$Q_{1000\ years}$	Q_{PMF}
Outflow Max.	Without Control	[m³/s]	284.29	303.10	317,35	330.38	369.74	408.80
	With Control	[m³/s]	115.00	132.60	150,50	172.70	110.50	142.00
The start time of the outflow discharge gate setting			3	3	3	3	2	2
Reservoir Lake	Without Control	[millions m³]	54,525,970.44	64,223,414.39	72,746,921.73	81,538,839.23	115,025,073.60	161,205,248.81
	With Control	[millions m³]	67,030,989.55	78,001,449.55	86,623,089.55	94,680,069.55	143,987,109.55	192,845,409.55
Reservoir Lake Water Level Elevation Max	Without Control	[m]	4.70	4.97	5.19	5.40	6.09	6.84
	With Control	[m]	5.05	5.32	5.52	5.69	6.58	7.28

The outflow and maximum lake elevation by using gate arrangement with several return periods design flood is as follows: $Q_{20years}$ = 115.00 m^3/s with the lake elevation is +5.05 m, $Q_{25years}$ = 132.60 m^3/s with the lake elevation is +5.32 m, $Q_{50years}$ = 150.50 m^3/s with the lake elevation is +5.52 m, $Q_{100years}$ = 172.70 m^3/s with the lake elevation is +5.69 m, $Q_{1000years}$ = 110.50 m^3/s with the lake elevation is +6.58 m, Q_{PMF} = 142.00 m^3/s with the lake elevation is +7.28 m.

3.3. Optimization of Gate Arrangement

To obtain the outflow by several scenarios of gate opening, there is carried out the optimization for every return period of flood and height of gate opening as presented in Table 8.

Table 8. Scenario of Gate Opening Due to Several Return Periods Flood Discharge

D [m]	Discharge Outflow Maximum Without Control (Existing Condition) [m³/s]						Reservoir Lake Water Level Elevation Max With Control [m]					
	$Q_{20\ years}$	$Q_{25\ years}$	$Q_{50\ years}$	$Q_{100\ years}$	$Q_{1000\ years}$	Q_{PMF}	$Q_{20\ years}$	$Q_{25\ years}$	$Q_{50\ years}$	$Q_{100years}$	$Q_{1000\ years}$	Q_{PMF}
0.40	98.077	103.124	106.785	110.070	120.067	129.959	5.118	5.426	5.659	5.875	6.573	7.323
0.45	109.218	114.940	119.094	122.822	134.163	145.378	5.084	5.391	5.624	5.840	6.539	7.290
0.50	120.153	126.548	131.199	135.373	148.077	160.634	5.053	5.359	5.592	5.808	6.507	7.259
0.55	130.879	137.958	143.110	147.740	161.810	175.726	5.023	5.328	5.561	5.777	6.476	7.229
0.60	141.390	149.159	154.821	159.915	175.366	190.654	4.995	5.299	5.531	5.747	6.446	7.200
0.65	151.712	160.160	166.337	171.897	188.755	205.422	4.969	5.272	5.503	5.719	6.418	7.172
0.70	161.840	170.964	177.660	183.691	201.973	220.033	4.944	5.245	5.476	5.692	6.390	7.145
0.75	171.771	181.583	188.793	195.301	215.019	234.483	4.920	5.220	5.450	5.666	6.364	7.119
0.80	181.507	192.023	199.735	206.730	227.896	248.784	4.898	5.196	5.425	5.641	6.338	7.094
0.85	191.052	202.272	210.500	217.978	240.605	262.936	4.877	5.173	5.401	5.616	6.313	7.069
0.90	200.401	212.335	221.099	229.044	253.151	276.935	4.856	5.151	5.379	5.593	6.290	7.045
0,95	209.573	222.208	231.512	239.929	265.528	290.786	4.837	5.130	5.357	5.571	6.267	7.022
1,00	218.559	231.911	241.743	250.651	277.745	304.486	4.818	5.110	5.336	5.549	6.244	7.000
1,05	227.359	241.424	251.808	261.213	289.803	318.036	4.801	5.091	5.316	5.529	6.222	6.978
1,10	235.985	250.756	261.689	271.594	301.693	331.441	4.784	5.072	5.297	5.509	6.201	6.956
1,15	244.441	259.919	271.396	281.820	313.430	344.699	4.767	5.054	5.278	5.489	6.180	6.935
1,20	252.724	268.890	280.934	291.864	325.007	357.807	4.753	5.037	5.260	5.471	6.160	6.915
1,25	260.835	277.706	290.291	301.748	336.423	370.777	4.738	5.020	5.242	5.453	6.140	6.895
1,30	268.807	286.334	299.485	311.457	347.686	383.592	4.724	5,005	5.226	5.436	6.122	6.876
1,35	276.617	294.806	308.502	321.010	358.789	396.277	4.713	4.989	5.209	5.418	6.103	6.857
1,40	284.286	303.102	317.355	330.381	369.740	408.805	4.701	4.975	5.194	5.403	6.085	6.838

The outflow and elevation of maximum lake without gate arrangement by several return periods design flood is as follows: $Q_{20years}$ = 284.29 m^3/s with the lake elevation is +4.70 m, $Q_{25years}$ = 303.10 m^3/s with the lake elevation is +4.97 m, $Q_{50years}$ = 317.35 m^3/s with the lake elevation is +5.19 m, $Q_{100years}$ = 330.38 m^3/s with the lake elevation is +5.40 m, $Q_{1000years}$ = 369.74 m^3/s with the lake elevation is +6.09 m, and Q_{PMF} = 408.80 m^3/s with the lake elevation is +6.84 m.

Based on the scenarios above, there is obtained the regression equation as the relation between maximum outflow and gate opening height and elevation of maximum reservoir water level and gate opening height as presented in Table 9.

Table 9. Equation of Outflow Discharge and Reservoir Elevation with Gate Opening Height

D [m]	Discharge Outflow Maximum Without Control (Existing Condition) [m³/s]						Reservoir Lake Water Level Elevation Max With Control [m]					
	$Q_{20\ years}$	$Q_{25\ years}$	$Q_{50\ years}$	$Q_{100\ years}$	$Q_{1000\ years}$	Q_{PMF}	$Q_{20\ years}$	$Q_{25\ years}$	$Q_{50\ years}$	$Q_{100years}$	$Q_{1000\ years}$	Q_{PMF}
m	217.140	230.475	240.359	249.325	276.494	303.358	4.816	5.106	5.331	5.544	6.236	6.990
n	0.847	0.859	0.868	0.876	0.897	0.914	-0.069	-0.071	-0.070	-0.068	-0.063	-0.056

The equation of maximum outflow and reservoir water level elevation for the scenario with gate opening $Out/Elv_{maks} = m\,xD^n$, parameter of m and n for maximum outflow as follows: $Q_{20years}$ (m = 217.140 and n = 0.847, $Q_{25years}$ (m = 230.475 and n = 0.859), $Q_{50years}$ (m = 240.359 and n = 0.868), $Q_{100years}$ (m = 249.325 and n = 0.876), $Q_{1000years}$ (m = 276.494 and n = 0.896), and Q_{PMF} (m = 303.358 and n = 0.914), however, for maximum elevation is as follows: $Q_{20years}$ (m = 4.816 and n = -0.069), $Q_{25years}$ (m = 5.106 and n = -0.071), $Q_{50years}$ (m = 5.331 and n = -0.070), $Q_{100years}$ (m = 5.543 and n = -0.068), $Q_{1000years}$ (m = 6.235 and n = -0.063), and Q_{PMF} (m = 6.990 and n = -0.056).

4. Conclusion

Limboto Lake in Gorontalo Province is a part of a Laguna that the estuary is in Bolango and Bone Rivers. The lake water is from 23 affluent that flow into the lake from northern, western, and southern watersheds, with the Limboto Lake area being 4,419.72 ha. The flood event that happens in Gorontalo Province mainly in Gorontalo Regency and Gorontalo City is due to the fact that there has not been a regulation related to the operation pattern for Limboto Lake, which the outlet of Limboto Lake is Tapodu River, the estuary in Bolango River, and the end estuary is in Bone River (Tomini Bay), so there has been optimized the Limboto Lake management as the flood control by gate opening simultaneously. Based on the optimization analysis result of storage and gate opening in Limboto Lake, it can be concluded that the outflow and elevation of the maximum lake without a gate arrangement for $Q_{1000years}$ = 369.74 m³/s with the lake elevation at +6.09 m, and Q_{PMF} = 408.80 m³/s with the lake elevation at +6.84 m. However, the outflow and maximum lake elevation by using the gate arrangement for $Q_{1000years}$ = 110.50 m³/s with the lake elevation at +6.58 m, Q_{PMF} = 142.00 m³/s with the lake elevation at +7.28 m. By the capacity of the Tapodu canal of 199.00 m³/s and the Limboto lake elevation of +7.00 m, the outflow without a gate arrangement can cause the Tapodu canal to overflow at a height of 0.28 cm. The equation of maximum outflow and reservoir water level elevation for the scenario with gate opening $Q_{1000years}$ (m = 6.235 and n = -0.063) and Q_{PMF} (m = 6.990 and n = -0.056).

5. Declarations

5.1. Author Contributions

Conceptualization, F.K. and L.M.L.; methodology, E.Y.; software, W.S.; validation, F.K., L.M.L., and W.S.; formal analysis, F.K.; investigation, F.K.; resources, E.Y.; data curation, F.K.; writing—original draft preparation, L.M.L.; writing—review and editing, E.Y.; visualization, W.S.; supervision, L.M.L.; project administration, F.K.; funding acquisition, F.K. All authors have read and agreed to the published version of the manuscript.

5.2. Data Availability Statement

The data presented in this study are openly available in the article.

5.3. Funding

The authors received no financial support for the research, authorship, and/or publication of this article.

5.4. Institutional Review Board Statement

Not applicable.

5.5. Informed Consent Statement

Not applicable.

5.6. Declaration of Competing Interest

The authors declare that there are no conflicts of interest concerning the publication of this manuscript. Furthermore, all ethical considerations, including plagiarism, informed consent, misconduct, data fabrication and/or falsification, double publication and/or submission, and redundancies have been completely observed by the authors.

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