

Assessment of the Condition of Limboto Lake as a Flood Control Storage

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Abstract: This research intends to evaluate the condition of Limboto Lake as a flood control storage in Gorontalo Province. Limboto Lake is located in Gorontalo Province as the part and a laguna that has the estuary in Bolango River and Bone River. The geological process as the tectonic movement has separated the laguna from the sea and formed the freshwater lake. The lake water comes from 23 affluents that flow into the lake from the northern, western, and southern catchment areas. The area of Limboto Lake is 4,410.72 ha. The methodology consists of literature study, collecting secondary data, analyzing secondary data, and evaluating the secondary data analysis. The result shows that the rainfall data availability, which is from six rainfall stations spread in the Limboto Lake catchment area, is still less (less than 20 years), so to complete and generate the rainfall data, the result of satellite observation, Global Precipitation Measurement (GPM), is used. The design flood due to some return periods is 1,553.2 m³/dt for Q_{25years}, 1,876.7 m³/s for Q_{50years}, 2,205.8 m³/s for Q_{100years}, 3,364 m³/s for Q_{1000years}, and 4,839.4 m³/s for Q_{PMF}. The result of flood routing in Limboto Lake is obtained that the condition of embankment is still safe if there is happened the design flood less than 1,000 years return period. However, if there is the PMF discharge, so it will lead to Limboto Lake embankment overtopping.

Keywords: Limboto Lake, flood control storage, catchment area.

林博托湖作为防洪蓄水池的状况评估

摘要：本研究旨在评估林博托湖作为哥伦打洛省防洪蓄水池的状况。林博托湖位于哥伦打洛省的一部分，是博兰戈河和博恩河入海口的一个泻湖。构造运动的地质过程使拉古纳与海洋分离，形成淡水湖。湖水来自23条支流，分别从北部、西部和南部流域流入该湖。林博托湖面积为4,410.72公顷。该方法包括文献研究、收集二手数据、分析二手数据和评估二手数据分析。结果表明，分布在林博托湖流域的6个雨量站的降雨数据可用性仍然较少（不到20年），因此要完成和生成降雨数据，需要卫星观测的结果，全球降水使用测量（毛利率）。部分重现期设计洪水量为问25年1,553.2立方米/分压、问50年1,876.7立方米/秒、问100年2,205.8立方米/秒、问1000年3,364立方米/秒、QPMF4,839.4立方米/秒。林博托湖洪水演算结果表明，如果发生小于1000年一遇的设计洪水，堤坝状况仍然安全。然而，如果有PMF排放

, 那么就会导致林博托湖堤坝漫溢。

关键词：林博托湖，防洪库，集水区。

1. Introduction

Flooding is one of the environmental problems [1, 2] that occur a lot in some watersheds in Indonesia. Simply, flood can be defined as a flow or inundation that causes loss for humans. Flood is related to the hydrology cycle, i.e., flood will occur if the volume of rainfall that enters is more than the water capacity that comes out, i.e., there is water surplus.

The storage management of a dam or lake can be used under the maximum and minimum conditions that depend on the regulation of reservoir operation [3, 4]. In the flood season, the water level of the reservoir cannot be left too low because the water is used in the downstream. The flood limit water level (FLWL) is a parameter of reservoir storage optimization [7]. The use of reservoir/lake storage in flood control is affected by inflow to a reservoir/lake, storage capacity, and river capacity in the downstream of a reservoir/lake. Following Zhou [8], the three-series reservoir operation is optimized for reducing floods in the points that are prone to flooding in the dam downstream. Lake storage is used for flood control in the research of Yang [9] in Hongze and Luoma Lakes. The result of the optimization scenario is that the flood storage capacity of Hongze Lake increased from 12.5 m to 13.0 m, and that of Luoma Lake - from 22.5 m to 23.5 m. The increase can also be used for fulfilling the water need in the downstream, which is 413 million m³.

This research intends to evaluate the condition of Limboto Lake as a flood control storage in Gorontalo Province. The shallowing speed of the lake is due to erosion from the rivers that have estuaries in the immense lake. In 1932, the average depth of Limboto Lake was 30 m with the area of 7,000 ha; in 1955, the

lake depth decreased to 16 m; in 1961, the average depth of Limboto Lake decreased to 10 m, and the area decreased to 4,250 ha. In 1990-2004, the average depth of Limboto Lake was 2.5 m, and the rest area was 3,000 ha. In the 52 years' time period, the Limboto Lake is decreasing 4,304 ha (62.06%) [10]. Decrease in the lake storage power causes Limboto Lake to experience floods in almost every rainy season. Besides being an income source for fishermen and a tourism facility, Limboto Lake also plays a role in flood control in Gorontalo Province. The storage of Limboto Lake is the retention of flooding of 23 affluents before being released into Bolango River that has the estuary in Bone River.

2. Materials and Methods

2.1. Condition

2.1.1. Catchment Area (DTA)

Limboto Lake is located in the Bolango-Limboto watershed. The area of the Bolango-Limboto watershed is 1,088 km². The boundary delineation of the Limboto Lake catchment area (DTA) is carried out using a digital elevation model (DEM). The national DEM published by the Spatial Information Institution (BIG) is in the shape-file (.shp) format with the 0.27-arcsecond (± 11.25 m) resolution. Through spatial analysis using GIS software, we can determine the area of each catchment area of Limboto Lake. To obtain the maximal result, the sub-DTA is divided into 79 sub-DTAs. Fig. 1 presents the map of the Limboto sub-watershed.

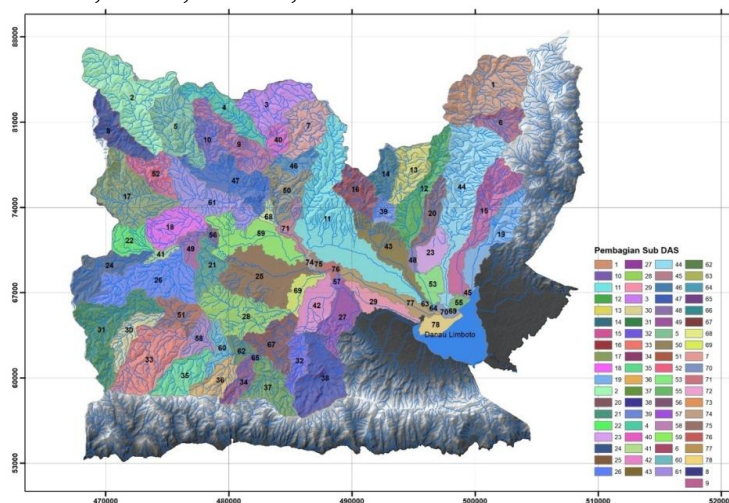


Fig. 1 Map of the Limboto Lake sub-watershed, catchment area

2.1.2. Land Cover

Based on the data from the Spatial Information Institution (BIG), the land-use map for 2019 is obtained in shape-file format. After overlaying the catchment

area boundary, it can be seen that the composition of land cover in the DTA boundary in Limboto Lake is dominated by bushes and dryland farming, as presented in Fig. 2.

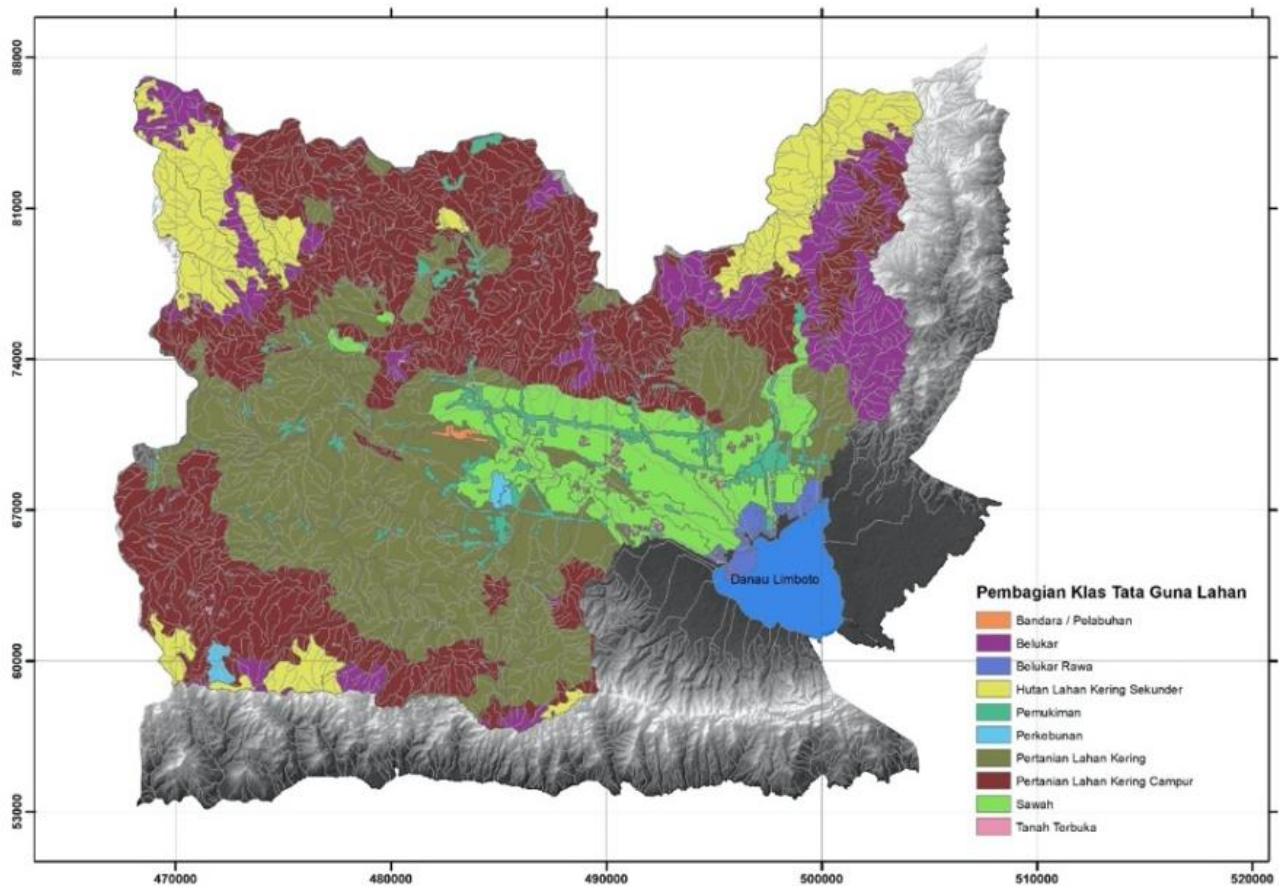


Fig. 2 The Limboto Lake catchment area land cover map

From the land cover map above, the composition of land cover in Limboto Lake is presented in Table 1.

Table 1 Land cover composition in the Limboto Lake watershed (Analysis results, 2023)

Klas Tata Guna Lahan	Luas(km2)	%
Hutan Lahan Kering Sekunder	73.39	9.9%
Belukar	68.94	9.3%
Tanah Terbuka	4.90	0.7%
Pertanian Lahan Kering Campur	263.21	35.6%
Perkebunan	3.00	0.4%
Pemukiman	24.55	3.3%
Belukar Rawa	5.99	0.8%
Pertanian Lahan Kering	220.52	29.9%
Sawah	73.05	9.9%
Bandara / Pelabuhan	0.81	0.1%

2.1.3. Capacity of the Lake

The shallowing speed of the lake is due to erosion from the rivers that have estuaries in the immense lake.

In 1932, the average depth of Limboto Lake was 30 m with the area of 7,000 ha; in 1955, the lake depth decreased to 16 m; in 1961, the average depth of Limboto Lake decreased to 10 m, and the area decreased to 4,250 ha. In 1990-2004, the average depth of Limboto Lake was 2.5 m, and the rest area was 3,000 ha. In the 52 years' time period, the Limboto Lake is decreasing 4,304 ha (62.06%) [10]. Decrease in the lake storage power causes Limboto Lake to experience floods in almost every rainy season. The areas always inundated during the rainy season are surrounded by Biyonga and Tapodu Rivers downstream and along the Alo Puhu River. Fig. 3 presents the bathymetry contour and topography map of Limboto Lake, 2020. However, Fig. 4 presents the relation between elevation (H) and volume of the lake (S), and Table 2 presents the elevation and inundation of Limboto Lake.

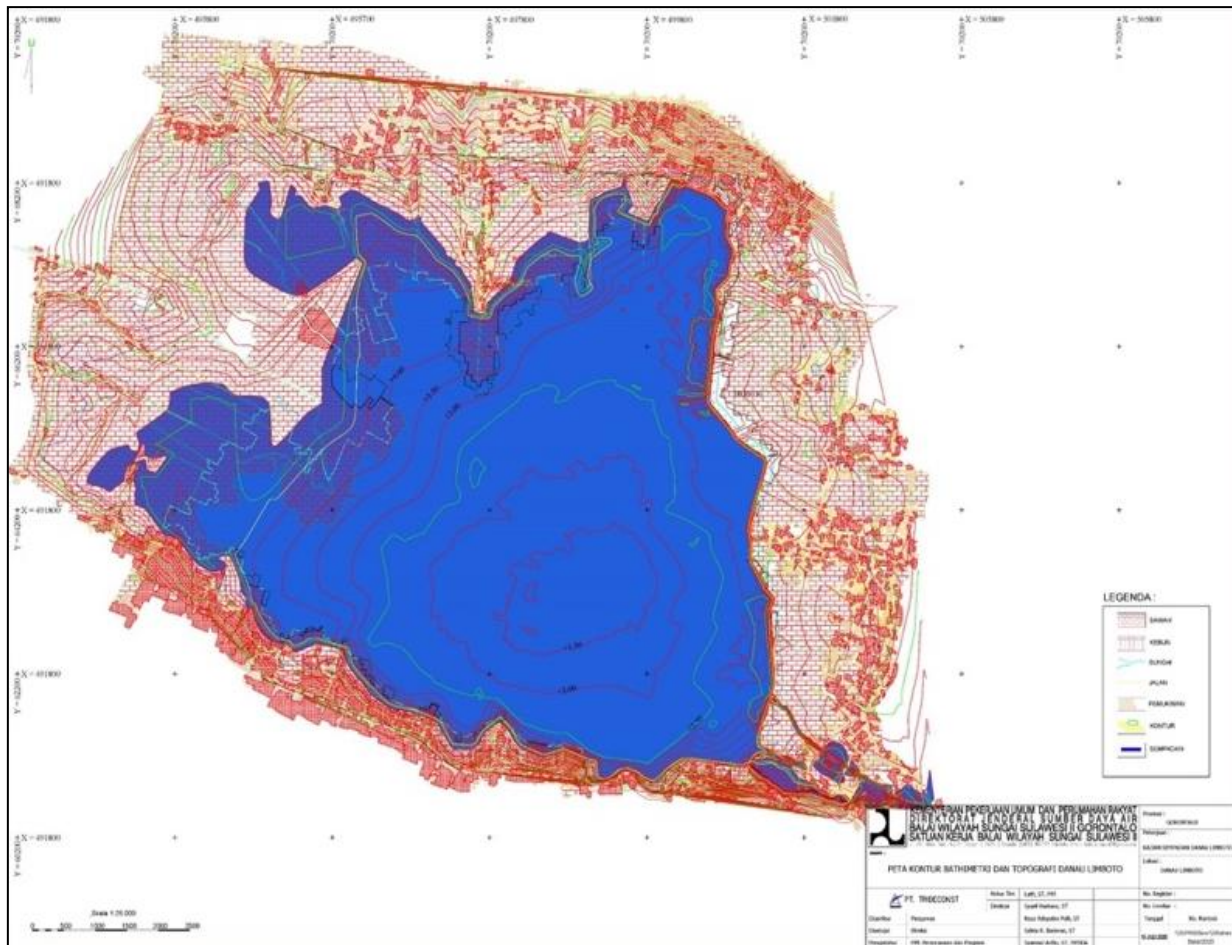


Fig. 3 Bathymetry contour and topography map of Limboto Lake, 2020

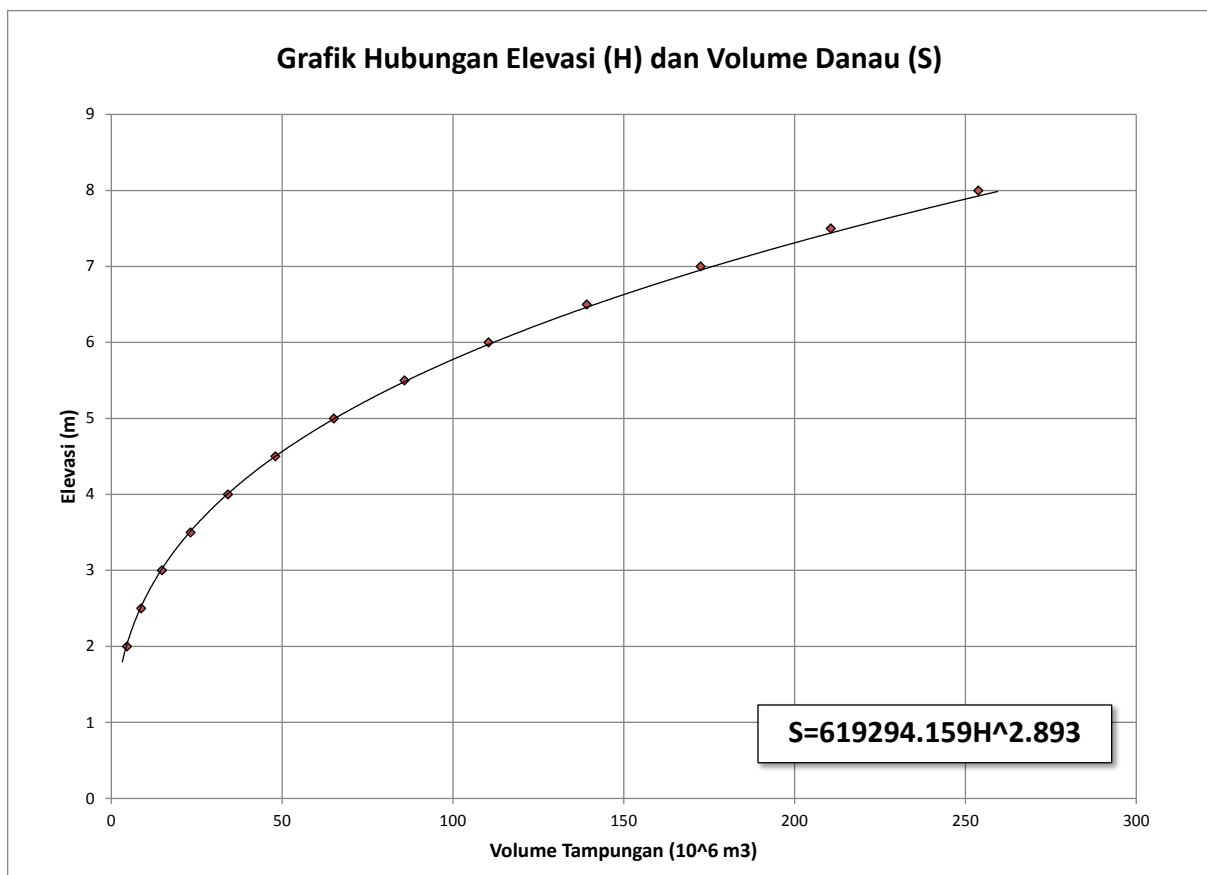


Fig. 4 Relationship between elevation (H) and volume of the lake (S)

Table 2 Elevation and inundation of Limboto Lake [10]

No	Elevation	ΔH	Inundation area		Volume		Category
	H (m)		m ²	ha	10 ³ m ³	million m ³	
1	1.50	0	2,181,395	218.139457	-	-	Low
2	2.00	0.5	6,788,289	678.828871	4,599.96	4,600	
3	2.50	1	13,833,488	1383.348754	8,772.17	8,772	
4	3.00	1.5	20,636,400	2063.639977	14,865.25	14,865	Normal
5	3.50	2	24,892,297	2489.229718	23,219.04	23,219	
6	4.00	2.5	27,411,855	2741.185466	34,167.29	34,167	
7	4.50	3	31,592,104	3159.210365	48,038.66	48,039	High
8	5.00	3.5	35,832,039	3583.203882	65,157.41	65,157	
9	5.50	4	39,940,392	3994.03924	85,843.95	85,844	
10	6.00	4.5	43,039,509	4303.950854	110,415.27	110,415	
11	6.50	5	45,892,320	4589.23198	139,185.26	139,185	
12	7.00	5.5	48,821,770	4882.177028	172,465.04	172,465	
13	7.50	6	51,105,973	5110.597251	210,563.16	210,563	
14	8.00	6.5	52,874,500	5287.45001	253,785.82	253,786	

From the bathymetry measurement in 2020, it can be seen that the capacity of Limboto Lake now is in the normal condition of 34,167,290 m³ and in flood condition of 85,843,950 m³.

3. Results

3.1. Analysis of the Rainfall Data

The hydrology analysis in this research uses rainfall data converted into discharge data. However, the

distribution of the six rainfall stations in the Limboto Lake watershed is presented in Table 3 and Fig. 5.

Table 3 Distribution of the rainfall stations and data availability

Rainfall station	Period of data	X	Y	Information
Ombulo	2017-2021	122.92	0.65	- daily format
Biyonga	2007-2021	123.04	0.70	- daily format
Iloponu	2012-2021	122.86	0.71	- daily format
Datahu	2007-2021	122.86	0.65	- daily format
Tabango	2007-2021	122.93	0.57	- daily format
Pilolalenga	2007-2021	122.87	0.60	- daily format

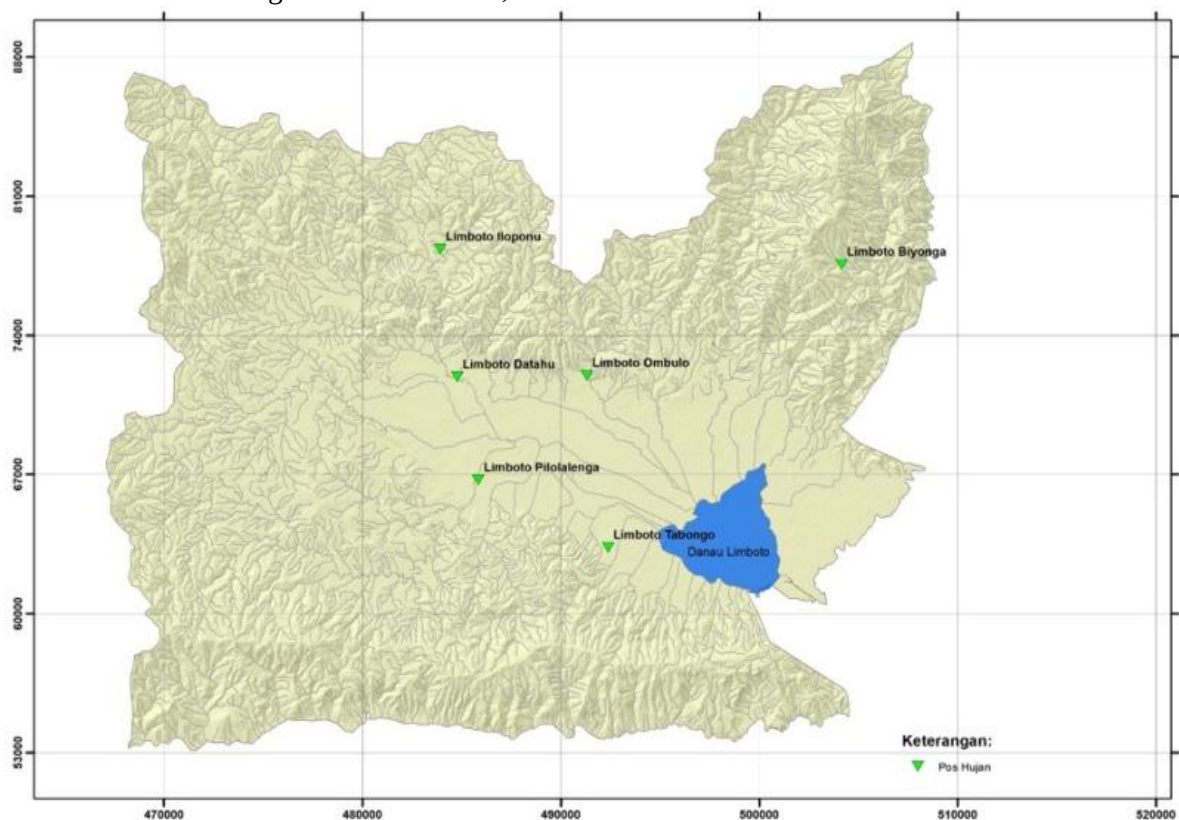


Fig. 5 Distribution of the rainfall stations in the Limboto Lake catchment area

Based on the Figure and Table above, by considering the density of rainfall stations, the Thiessen polygon is used in analyzing the rainfall data on Limboto Lake. However, in analyzing the high and low flows, the calibration is not carried out because there are no hourly discharge data there.

3.2. Satellite Rainfall Data

Because the rainfall data in some rainfall stations in the Limboto catchment area are not complete (less than 10 years), so to complete the data availability can be

considered to use the rainfall satellite that has the spatial resolution and high temporal, scope of area is wide, fast access, and economics. This research uses satellite rainfall data from Global Precipitation Measurement (GPM) with the $0.10^{\circ} \times 0.10^{\circ}$ spatial resolution. The observation period for GPM data is from January 1, 2001, to December 31, 2022, so the obtained period with complete data is from 2001 to 2022 (21 years). Fig. 6 presents the grid distribution of the satellite rainfall data.

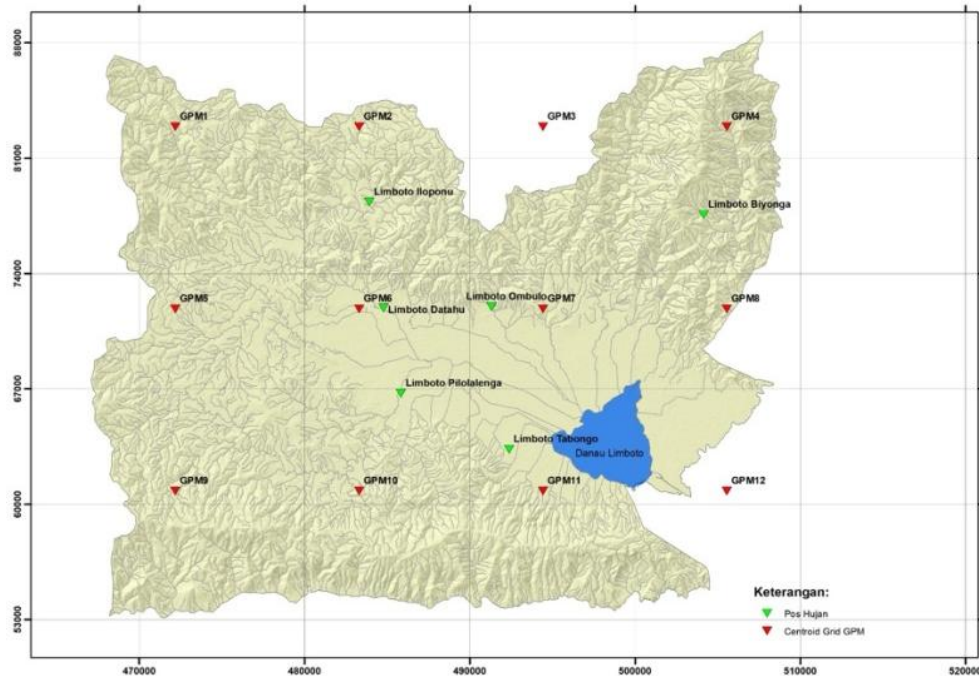


Fig. 6 Grid distribution of the satellite rainfall data to the Limboto Lake catchment area

3.3. Validation and Correction of the Satellite Rainfall Data

Before the GPM data are used, the validation and correction of HHMT (maximum daily rainfall) data are

carried out at the observation station. Table 4 presents the results of the rainfall station data validation and correction to the satellite data.

Table 4 Results of the rainfall station data validation and correction to the satellite data

Before being corrected												
Error Daily	GPM_1	GPM_2	GPM_3	GPM_4	GPM_5	GPM_6	GPM_7	GPM_8	GPM_9	GPM_10	GPM_11	GPM_12
Ombulo	0.01168	0.01222	0.01293	0.01383	0.01021	0.01063	0.01080	0.01186	0.01053	0.01057	0.01083	0.01175
Biyonga	0.01083	0.01095	0.01094	0.01108	0.01159	0.01141	0.01148	0.01167	0.01260	0.01243	0.01270	0.01314
Iloponu	0.01101	0.01146	0.01162	0.01209	0.01143	0.01147	0.01161	0.01199	0.01198	0.01191	0.01233	0.01299
Datahu	0.01147	0.01186	0.01242	0.01324	0.01071	0.01110	0.01130	0.01202	0.01133	0.01120	0.01148	0.01233
Tabongo	0.01177	0.01236	0.01302	0.01392	0.01028	0.01080	0.01109	0.01213	0.01028	0.01044	0.01068	0.01161
Pilolalenga	0.01223	0.01255	0.01313	0.01397	0.01146	0.01180	0.01213	0.01273	0.01209	0.01187	0.01233	0.01316
To be corrected												
Error Daily	GPM_1	GPM_2	GPM_3	GPM_4	GPM_5	GPM_6	GPM_7	GPM_8	GPM_9	GPM_10	GPM_11	GPM_12
Ombulo	0.01125	0.01174	0.01244	0.01337	0.00991	0.01042	0.01055	0.01146	0.01034	0.01024	0.01057	0.01142
Biyonga	0.01067	0.01076	0.01079	0.01088	0.01161	0.01154	0.01153	0.01150	0.01265	0.01249	0.01271	0.01317
Iloponu	0.01074	0.01110	0.01115	0.01159	0.01122	0.01129	0.01140	0.01169	0.01188	0.01175	0.01211	0.01281
Datahu	0.01122	0.01153	0.01199	0.01278	0.01055	0.01091	0.01110	0.01174	0.01114	0.01104	0.01134	0.01203
Tabongo	0.01134	0.01191	0.01253	0.01346	0.00989	0.01046	0.01072	0.01174	0.00999	0.01077	0.01042	0.01120
Pilolalenga	0.01197	0.01218	0.01262	0.01349	0.01127	0.01159	0.01190	0.01248	0.01187	0.01177	0.01212	0.01283
Deviation												
Error Daily	GPM_1	GPM_2	GPM_3	GPM_4	GPM_5	GPM_6	GPM_7	GPM_8	GPM_9	GPM_10	GPM_11	GPM_12
Ombulo	0.04%	0.05%	0.05%	0.05%	0.03%	0.02%	0.03%	0.04%	0.02%	0.03%	0.03%	0.03%
Biyonga	0.02%	0.02%	0.02%	0.02%	0.00%	-0.01%	0.00%	0.02%	0.00%	-0.01%	0.00%	0.00%
Iloponu	0.03%	0.04%	0.05%	0.05%	0.02%	0.02%	0.02%	0.03%	0.01%	0.02%	0.02%	0.02%
Datahu	0.02%	0.03%	0.04%	0.05%	0.02%	0.02%	0.02%	0.03%	0.02%	0.02%	0.01%	0.03%
Tabongo	0.04%	0.04%	0.05%	0.05%	0.04%	0.03%	0.04%	0.04%	0.03%	-0.03%	0.03%	0.04%
Pilolalenga	0.03%	0.04%	0.05%	0.05%	0.02%	0.02%	0.02%	0.03%	0.02%	0.01%	0.02%	0.03%

Based on Table 4, the GPM grid data have a small deviation. From the validation and correction results, we obtained the correction coefficients (Tables 5-7).

Table 5 Satellite Rainfall Data Validation and Correction Results

Range of rainfall (mm)			Koefisien
0	-	0.00	0
0.00	-	3.00	0.7
3.00	-	10.00	0.75
10.00	-	15.00	0.78
15.00	-	30.00	0.8
30.00	-	50.00	0.82
		> 120	0.85

Table 6 Correction of the yearly maximum daily rainfall data

	GPM1	GPM2	GPM3	GPM4	GPM5	GPM6	GPM7	GPM8	GPM9	GPM10	GPM11	GPM12	OMBULO	BIYONGA	ILOPUNU	DATAHU	TABONGO	PILOLALENGA
GPM1	1.00	0.89	0.80	0.76	0.81	0.84	0.82	0.74	0.76	0.74	0.74	0.68	0.59	0.60	0.63	0.58	0.57	0.55
GPM2		1.00	0.88	0.79	0.82	0.89	0.86	0.78	0.77	0.80	0.78	0.74	0.65	0.63	0.64	0.63	0.64	0.60
GPM3			1.00	0.85	0.81	0.83	0.89	0.84	0.77	0.76	0.76	0.73	0.73	0.66	0.64	0.64	0.64	0.62
GPM4				1.00	0.76	0.76	0.86	0.85	0.72	0.71	0.75	0.71	0.66	0.67	0.73	0.58	0.56	0.60
GPM5					1.00	0.90	0.87	0.81	0.87	0.84	0.82	0.78	0.68	0.57	0.62	0.61	0.64	0.64
GPM6						1.00	0.89	0.83	0.84	0.86	0.85	0.81	0.73	0.63	0.68	0.61	0.68	0.61
GPM7							1.00	0.91	0.81	0.82	0.83	0.81	0.71	0.67	0.67	0.67	0.66	0.65
GPM8								1.00	0.78	0.77	0.82	0.83	0.60	0.64	0.68	0.60	0.61	0.61
GPM9									1.00	0.85	0.79	0.75	0.64	0.53	0.54	0.60	0.63	0.59
GPM10										1.00	0.89	0.83	0.63	0.55	0.62	0.59	0.67	0.61
GPM11											1.00	0.89	0.62	0.56	0.63	0.58	0.65	0.55
GPM12												1.00	0.63	0.56	0.64	0.56	0.64	0.53
OMBULO													1.00	0.53	0.72	0.84	0.77	0.74
BIYONGA														1.00	0.70	0.63	0.55	0.65
ILOPUNU															1.00	0.79	0.61	0.63
DATAHU																1.00	0.61	0.70
TABONGO																	1.00	0.63
PILOLALENGA																		1.00

Table 7 Grid selection and rainfall data filtering results

	GPM1	GPM2	GPM3	GPM4	GPM5	GPM6	GPM7	GPM8	GPM9	GPM10	GPM11	GPM12	OMBULO	BIYONGA	ILOPUNU	DATAHU	TABONGO	PILOLALENGA	CEK	KET				
GPM1	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	1	1	1	4 NO				
GPM2		0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0 OK				
GPM3			0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0 OK				
GPM4				0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	1	3 NO				
GPM5					0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	1 NO				
GPM6						0	0	0	0	0	0	0	0	0	0	0	0	0	0	0 OK				
GPM7							0	0	0	0	0	0	0	0	0	0	0	0	0	0 OK				
GPM8								0	0	0	0	0	1	0	0	0	0	0	0	1 NO				
GPM9									0	0	0	0	0	0	1	1	0	0	1	3 NO				
GPM10										0	0	0	0	0	1	0	1	0	0	2 NO				
GPM11											0	0	0	0	1	0	1	0	1	3 NO				
GPM12												0	0	0	1	0	1	0	1	3 NO				
OMBULO													0	1	0	0	0	0	0	1 NO				
BIYONGA															0	0	0	1	0	1 NO				
ILOPUNU																0	0	0	0	0	0	0	0	0 OK
DATAHU																			0	0	0	0	0	0 OK
TABONGO																					0	0	0	0 OK
PILOLALENGA																							0	0 OK

3.4. Design Rainfall

The design rainfall is obtained from the daily rainfall, which is analyzed using the rainfall frequency probability. The following methods can be used for this analysis: a) E.J. Gumbel, b) Log Pearson III, c) Iwai-Kadoya, and d) generalized extreme value (GEV). Then, the design rainfall analysis is carried out using Hydrognomon software. Table 8 presents the design rainfall of the GPM2 station.

Table 8 The GPM 2 station design rainfall

No	Return period	Rainfall (mm)
1	1.01	14.2
2	2	105.3
3	5	138.2
4	10	155.3
5	20	169.5
6	25	173.7
7	50	185.5
8	100	196.2
9	1000	226.0

Based on testing goodness of fit using the chi square and Kolmogorov-Smirnov tests, the normal distribution has the greatest reliability. Table 9 presents the recapitulation of the design rainfall at each station.

Table 9 Recapitulation of the design rainfall at each station

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	225.03	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

3.5. Probable Maximum Precipitation (PMP)

PMP in the Limboto Lake watershed is analyzed

using the Hershfield method. This method is suitable for watershed area < 1,000 km². The formulation of the

Hershfield method is as follows [6]:

$$X_{PMP} = \bar{X} + K \cdot S$$

where:

X_{PMP} - maximum rainfall that might happen;

K - coefficient factor of the Hershfield method;

S - standard deviation;

$\bar{X}$ - mean of rainfall/flood.

Table 10 presents the design rainfall of the Limboto Lake catchment area.

Table 10 Design rainfall of the Limboto Lake catchment area

No	Rainfall station	Distribution	1.01	2	5	10	20	25	50	100	1000	PMP
1	Limboto Datau	Log Normal	28.61	72.83	102.08	121.77	140.87	146.98	165.98	185.15	251.52	486.48
2	Limboto Tabongo	Normal	31.03	73.40	88.70	96.70	103.31	105.24	110.75	115.70	129.60	427.74
3	Limboto Pilolalenga	Log Normal	40.34	71.25	87.49	97.41	106.44	109.23	117.61	125.70	151.47	303.46
4	GPM 2	Normal	14.25	105.27	138.15	155.34	169.53	173.66	185.50	196.15	225.99	767.58
5	GPM 3	Log Normal	31.22	89.30	130.54	159.19	187.54	196.71	225.52	225.03	359.95	820.45
6	GPM 6	Log Normal	28.25	89.12	134.95	167.64	200.53	211.27	245.32	280.61	408.95	905.86
7	GPM 7	Normal	2.56	88.68	119.79	136.05	149.48	153.39	164.59	174.67	202.91	726.18
	Mean of DTA		24.25	86.40	118.20	138.10	156.70	162.55	180.47	198.27	258.57	666.75

4. Discussion

4.1. Design Flood

Because flood observed discharge is not available, so for analysis of flood discharge is used the unit hydrograph approach that is direct run-off hydrograph produced by more even rainfall in the whole catchment area due to the constant intensity in time unit. The basic concept in unit hydrographs is that different unit rainfall will produce almost the same distribution curve. The design flood modeling will be carried out using the synthetic unit hydrograph method, HSS SCS-CN. The components of the design flood model are [5]: a) loss method (effective rainfall); b) transform method (flood hydrograph); and c) baseflow.

4.1.1. Effective Rainfall

The effective rainfall is affected by the area condition, watershed characteristics, and rainfall into the river region. Effective rainfall is obtained from the difference between the total rainfall and precipitation loss. The precipitation loss is calculated using the infiltration approach. However, infiltration is the basic component of loss in precipitation that affects the value, timing, and distribution of surface runoff. The capacity of infiltration is the maximum speed in a piece of land that can absorb water under certain conditions and in a certain time. The analysis of effective rainfall

assumes that the groundwater content in a watershed is homogenic. In this case, we will use the SCS-CN method. The Soil Conservation Service-Curve Number (SCS-CN) is a model developed by the Natural Resource Conservation Service-Curve Number (NRCS-CN) to estimate runoff in a river or sub-river region. The SCS-CN method is a simple model, easy to predict, and stable. However, in its usage for a long time, the value of infiltration will be close to zero and not constant. The parameter needed in the analysis of loss using this method is the curve number, which is differentiated from the land cover, soil classification based on the Hydrology Soil Group (HSG), and rainfall. The value of CN is between 35 and 100. CN = 35 represents a region that has high enough infiltration ability; however, CN = 100 shows that all the rainfall becomes a direct runoff. The parameter input of the effective rainfall method using SCS-CN consists of the CN value and initial abstraction (Ia).

- CN Value

By using the software of HWSO Viewer (Harmonized World Soil Database), it can be seen that the Limboto Lake catchment area enters the soil group AC-Acrisols in which the soil texture is dominated by loam with a mix of clay and sandy clay loam. The composite CN values for the Limboto Lake catchment area are presented in Table 11.

Table 11 The composite CN values

S	A (km2)	A x CN	CN	S	A (km2)	A x CN	CN	S	A (km2)	A x CN	CN	S	A (km2)	A x CN	CN
S 1	26.72	1533.5	57	S 27	10.37	720.5	69	S 44	33.88	2282.6	67	S 62	3.17	219.0	69
S 10	6.42	444.1	69	S 28	17.60	1233.7	70	S 45	0.21	14.5	69	S 63	0.28	19.7	69
S 11	47.80	3339.2	70	S 29	6.57	461.1	70	S 46	4.52	317.2	70	S 64	0.27	18.9	70
S 12	15.24	986.6	65	S 3	15.24	1064.8	70	S 47	15.99	1103.4	69	S 65	0.32	22.2	69
S 13	10.16	643.3	63	S 30	7.27	501.9	69	S 48	1.30	102.9	79	S 66	0.01	0.4	69
S 14	6.48	424.5	66	S 31	21.71	1457.2	67	S 49	4.01	282.5	70	S 67	11.13	773.2	69
S 15	19.07	1261.8	66	S 32	5.93	409.0	69	S 5	17.48	1155.7	66	S 68	1.68	122.9	73
S 16	9.08	625.1	69	S 33	21.59	1455.2	67	S 50	9.96	695.5	70	S 69	4.69	353.0	75
S 17	22.23	1471.7	66	S 34	5.59	385.2	69	S 51	7.50	521.0	69	S 7	12.09	823.6	68
S 18	11.24	784.3	70	S 35	10.54	651.1	62	S 52	8.01	504.7	63	S 70	0.11	8.2	71
S 19	13.43	878.7	65	S 36	6.64	429.3	65	S 53	4.81	354.7	74	S 71	4.33	312.4	72
S 2	29.20	1746.5	60	S 37	10.87	747.3	69	S 55	0.11	7.6	69	S 73	0.01	0.7	69
S 20	7.36	508.4	69	S 38	19.02	1287.7	68	S 56	1.44	102.8	71	S 74	0.77	52.9	69
S 21	7.79	543.1	70	S 39	3.31	233.5	70	S 57	0.67	46.7	70	S 75	1.14	79.2	69
S 22	6.64	461.2	70	S 4	10.83	744.9	69	S 58	5.42	374.1	69	S 76	1.53	106.1	70
S 23	6.21	442.8	71	S 40	4.23	299.9	71	S 59	16.75	1192.6	71	S 77	6.81	474.4	70
S 24	11.33	788.3	70	S 41	1.82	130.9	72	S 6	7.29	474.3	65	S 78	1.45	100.2	69
S 25	22.69	1617.8	71	S 42	9.49	671.2	71	S 60	5.05	348.6	69	S 8	10.69	591.8	55
S 26	22.95	1589.4	69	S 43	12.82	927.0	72	S 61	17.07	1178.0	69	S 9	12.94	916.8	71

- Initial Abstraction (I_a)

Initial abstraction is calculated using the following formula:

$$I_a = 0.2 \times S$$

$$S = (25400/CN) - 254$$

where:

I_a – initial abstraction;

S – potential retention maximum m ;

CN – curve number.

Therefore, I_a is obtained, 11.63-35.83 mm.

4.1.2. Flood Hydrograph

The SCS method is an empirical approach that is used for analyzing the direct runoff from rainfall, beginning from watersheds as small agricultural areas, forests, and urban areas, and can combine some watershed characteristics. The use of the SCS method in determining the effective rainfall expresses that the variable of curve number (CN) is the other watershed characteristic function such as soil, cover crop, land use, moisture, and the method of land preparation. The equation for discharge analysis using the HSS SCS-CN method is as follows:

$$Q_p = (0.0021 \times Q \times A) / T_p$$

where:

Q_p - speed of the surface peak flow (m^3/s);

Q - volume of the surface flow (m);

A - watershed area (m^2);

T_p - time to the peak (s).

The design flood modeling by using the SCS-CN method is then carried out using HEC-HMS 4.10 software. However, the effective rainfall in SCS-CN is transformed into the difference between rainfall and the loss in the form of flow/surface runoff by using parameter input of lag time (minute) with the following equation:

$$T_{lag} = 0.6 \times T_c$$

where:

T_{lag} - time lag (minutes);

T_c - concentration time (minutes).

However, T_c is analyzed by using the Kirpich formula as follows:

$$T_c = 0.0195 \times L^{0.77} \times S^{-0.385}$$

where:

L - length of the main river (m);

S - speed of the main river stream (m/m).

Therefore, Tables 12 and 13 present the concentration time and time lag.

Table 12 Concentration time

No	Sub Basin	Tc (minute)	No	Sub Basin	Tc (minute)	No	Sub Basin	Tc (minute)	No	Sub Basin	Tc (minute)
1	S1	61.32	21	S28	124.81	41	S46	46.02	61	S64	31.91
2	S10	41.43	22	S29	260.83	42	S47	119.60	62	S65	17.29
3	S11	249.88	23	S3	93.33	43	S48	82.23	63	S66	110.26
4	S12	97.98	24	S30	70.84	44	S49	75.11	64	S67	28.75
5	S13	81.35	25	S31	126.22	45	S5	102.64	65	S68	110.03
6	S14	47.16	26	S32	53.27	46	S50	82.76	66	S69	4497.02
7	S15	104.10	27	S33	91.07	47	S51	82.08	67	S7	88.18
8	S16	51.90	28	S34	47.66	48	S52	70.38	68	S70	10741.05
9	S17	80.34	29	S35	53.73	49	S53	115.64	69	S71	62.85
10	S18	84.19	30	S36	33.89	50	S54	1.90	70	S72	900.71
11	S19	91.06	31	S37	58.86	51	S55	10145.23	71	S73	17.33
12	S2	105.61	32	S38	72.85	52	S56	43.48	72	S74	61.11
13	S20	63.17	33	S39	35.05	53	S57	134.58	73	S75	72.34
14	S21	71.50	34	S4	84.07	54	S58	47.24	74	S76	52.07
15	S22	68.74	35	S40	39.32	55	S59	150.17	75	S77	304.02
16	S23	115.72	36	S41	61.22	56	S6	33.20	76	S78	189.58
17	S24	83.65	37	S42	83.88	57	S60	64.01	77	S79	43.65
18	S25	113.08	38	S43	111.66	58	S61	133.07	78	S8	59.28
19	S26	118.01	39	S44	182.67	59	S62	62.15	79	S9	100.51
20	S27	65.32	40	S45	48.07	60	S63	24.35			

Table 13 Time lag

No	Sub Basin	TLag (minute)	No	Sub Basin	TLag (minute)	No	Sub Basin	TLag (minute)	No	Sub Basin	TLag (minute)
1	S1	36.79	21	S28	74.89	41	S46	36.73	61	S64	19.15
2	S10	24.86	22	S29	156.50	42	S47	50.33	62	S65	10.37
3	S11	149.93	23	S3	56.00	43	S48	66.99	63	S66	66.16
4	S12	58.79	24	S30	42.50	44	S49	109.60	64	S67	17.25
5	S13	48.81	25	S31	75.73	45	S5	28.84	65	S68	66.07
6	S14	28.30	26	S32	31.96	46	S50	27.61	66	S69	2698.21
7	S15	62.46	27	S33	56.64	47	S51	71.76	67	S7	52.91
8	S16	31.14	28	S34	28.60	48	S52	49.31	68	S70	6444.63
9	S17	48.21	29	S35	32.24	49	S53	45.06	69	S71	37.71
10	S18	50.52	30	S36	20.13	50	S54	61.59	70	S72	540.42
11	S19	54.64	31	S37	35.31	51	S55	49.66	71	S73	10.40
12	S2	63.37	32	S38	43.71	52	S56	49.25	72	S74	36.67
13	S20	37.90	33	S39	21.03	53	S57	42.23	73	S75	43.40
14	S21	42.90	34	S4	50.44	54	S58	69.38	74	S76	31.24
15	S22	41.24	35	S40	23.59	55	S59	1.14	75	S77	182.41
16	S23	69.43	36	S41	36.73	56	S6	6087.14	76	S78	113.75
17	S24	50.19	37	S42	50.33	57	S60	26.09	77	S79	26.19
18	S25	67.85	38	S43	66.99	58	S61	80.75	78	S8	35.57
19	S26	70.81	39	S44	109.60	59	S62	28.34	79	S9	60.31
20	S27	39.19	40	S45	28.84	60	S63	90.10			

4.1.3. Baseflow Model

The component of baseflow is analyzed using the exponential recession method. The parameters used in the recession method are initial discharge, recession constant, and threshold flow. The initial flow is the initial baseflow that can be analyzed from observation data, and if the observation data are not found, this value can be determined by trial and error by comparing to the result of flood modeling with the other transform method. The initial type in this

modeling is selected as the discharge per area with the following equation of initial discharge:

$$Q_b = 0.4715 \times A^{0.6444} \times D^{0.943}$$

where:

Q_b - initial discharge (m³/s);

A - watershed area (km²);

D - drainage density (km/km²);

L - river length (km).

Therefore, Table 14 presents the baseflow.

Table 14 Base flow

Sub Basin	Area (km2)	D (km/km2)	Initial Discharge (m3/s)	Sub Basin	Area (km2)	D (km/km2)	Initial Discharge (m3/s)
S1	26.72	0.26	1.11	S46	4.52	0.37	0.49
S10	6.42	0.11	0.19	S47	15.99	0.54	1.56
S11	47.80	0.45	2.66	S48	1.30	2.06	1.10
S12	15.24	0.56	1.58	S49	4.01	0.79	0.93
S13	10.16	0.37	0.81	S5	17.48	0.50	1.54
S14	6.48	0.15	0.26	S50	9.96	0.76	1.60
S15	19.09	0.58	1.89	S51	7.50	0.47	0.85
S16	9.08	0.25	0.54	S52	8.01	0.91	1.65
S17	22.23	0.26	0.99	S53	4.81	0.65	0.87
S18	11.24	0.17	0.43	S54	0.00	11.47	0.08
S19	13.47	0.52	1.35	S55	1.45	1.17	0.70
S2	29.20	0.38	1.66	S56	1.44	1.81	1.04
S20	7.36	0.28	0.52	S57	0.67	1.78	0.63
S21	7.79	0.18	0.36	S58	5.42	0.63	0.90
S22	6.64	0.05	0.09	S59	16.75	0.59	1.76
S23	6.21	0.20	0.34	S6	7.29	0.33	0.60
S24	11.33	0.52	1.21	S60	5.05	0.46	0.65
S25	22.69	0.27	1.03	S61	17.08	0.62	1.86
S26	22.95	0.40	1.49	S62	3.17	0.93	0.93
S27	10.37	0.12	0.29	S63	0.29	2.31	0.47
S28	17.61	0.46	1.43	S64	0.36	1.71	0.40
S29	6.57	0.44	0.73	S65	0.32	3.28	0.70
S3	15.24	0.32	0.93	S66	11.14	0.69	1.56
S30	7.27	0.44	0.78	S67	1.68	1.32	0.86
S31	21.71	0.48	1.71	S68	4.69	1.15	1.45
S32	5.93	0.16	0.26	S69	0.05	0.42	0.03
S33	21.59	0.29	1.07	S7	12.09	0.39	0.96
S34	5.59	0.22	0.34	S70	1.19	1.05	0.55
S35	10.54	0.33	0.76	S71	4.33	1.04	1.26
S36	6.64	0.19	0.34	S72	0.01	3.82	0.06
S37	10.87	0.25	0.59	S73	0.13	1.81	0.22
S38	19.02	0.32	1.09	S74	0.77	2.01	0.77
S39	3.32	0.93	0.95	S75	1.14	0.85	0.44
S4	10.83	0.47	1.07	S76	1.53	1.13	0.69
S40	4.23	0.78	0.95	S77	6.81	1.40	2.23
S41	1.82	1.51	1.02	S78	3.80	0.65	0.74
S42	9.49	0.63	1.30	S79	0.37	1.43	0.35
S43	12.82	0.76	1.88	S8	10.69	0.44	0.99
S44	34.06	0.70	3.26	S9	12.94	0.41	1.06
S45	0.41	2.41	0.61				

The recession constant is the ratio between the constant flows now and yesterday, which has the value from 0 until 1, in this case is used 0.20. However, the threshold flow is the separated threshold value between

run-off flow and baseflow. In this case, we used ratio to the peak with 0.25. The results of flood modeling using the HSS SCS-CN method are presented in Table 15 and Fig. 7.

Table 15 Flood modeling results

Time	Q20th	Q25th	Q50th	Q100th	Q1000	QPMF	Time	Q20th	Q25th	Q50th	Q100th	Q1000	QPMF
0:00	75.3	75.3	75.3	75.3	75.3	75.3	1:00	46	66.4	57.5	74.2	104.9	142.7
1:00	77.5	78.5	78.1	79	80.5	82.4	2:00	43.3	62.4	54.1	69.8	98.6	134
2:00	86.5	91.8	89.3	94.9	110.5	142	3:00	40.8	58.7	50.9	65.6	92.7	125.9
3:00	115	150.5	132.6	172.7	283.7	511.2	4:00	38.4	55.2	47.9	61.7	87.2	118.3
4:00	214.7	375.9	294.4	474.8	929	1717.1	5:00	36.2	52	45.1	58.1	82	111.1
5:00	481.7	922	708	1161.5	2111.5	3484.3	6:00	34.1	48.9	42.5	54.7	77.1	104.5
6:00	875	1557.8	1241.3	1890	3094.3	4679.5	7:00	32.1	46.1	40.1	51.5	72.5	98.2
7:00	1154.7	1876.7	1553.2	2205.8	3364	4839.4	8:00	30.3	43.4	37.7	48.5	68.2	92.4
8:00	1189.7	1812.9	1536.5	2092	3069.1	4293.4	9:00	28.6	40.9	35.6	45.6	64.2	86.9
9:00	1064.7	1571.8	1348.1	1796.1	2568.2	3524.2	10:00	27	38.6	33.5	43	60.4	81.7
10:00	898.7	1294.6	1121.8	1468.4	2063.5	2799.6	11:00	25.4	36.3	31.6	40.5	56.9	76.9
11:00	741	1053	915.6	1191.1	1660.5	2234.1	12:00	24	34.3	29.8	38.2	53.6	72.4
12:00	615.5	869.8	756.7	980.5	1355.8	1818.4	13:00	22.7	32.3	28.1	36	50.5	68.1
13:00	523.8	735.7	639.7	826.3	1144.9	1544.1	14:00	21.4	30.5	26.6	34	47.6	64.2
14:00	456.1	640.4	556.4	717.4	1000	1358.5	15:00	20.2	28.8	25.1	32	44.8	60.4
15:00	404.5	571.1	495.1	640.7	903.5	1239.9	16:00	19.1	27.2	23.7	30.2	42.3	56.9
16:00	362.8	517.9	447.5	584.5	831.6	1145.6	17:00	18.1	25.6	22.4	28.5	39.9	53.7
17:00	329.8	475.9	410	539.8	773	1069.8	18:00	17.1	24.2	21.1	27	37.6	50.6
18:00	303.4	442.3	379.8	503.2	722.6	1000.1	19:00	16.2	22.9	20	25.5	35.5	47.7
19:00	284.3	416.1	356.9	473.3	679.5	941.6	20:00	15.3	21.6	18.9	24.1	33.5	45
20:00	267.9	393.4	336.9	447.4	641.3	890.6	21:00	14.5	20.5	17.9	22.8	31.7	42.5
21:00	255.1	376.4	321.6	427.1	610.6	848.2	22:00	13.8	19.4	16.9	21.5	29.9	40.1
22:00	243.4	359.5	307.3	406.4	580.5	804.1	23:00	13.1	18.3	16	20.4	28.3	37.9
23:00	231.7	341.7	292.6	384.9	550.3	758.8	0:00	12.4	17.4	15.2	19.3	26.7	35.8
0:00	219	322.2	276.7	361.9	518.2	712.4							

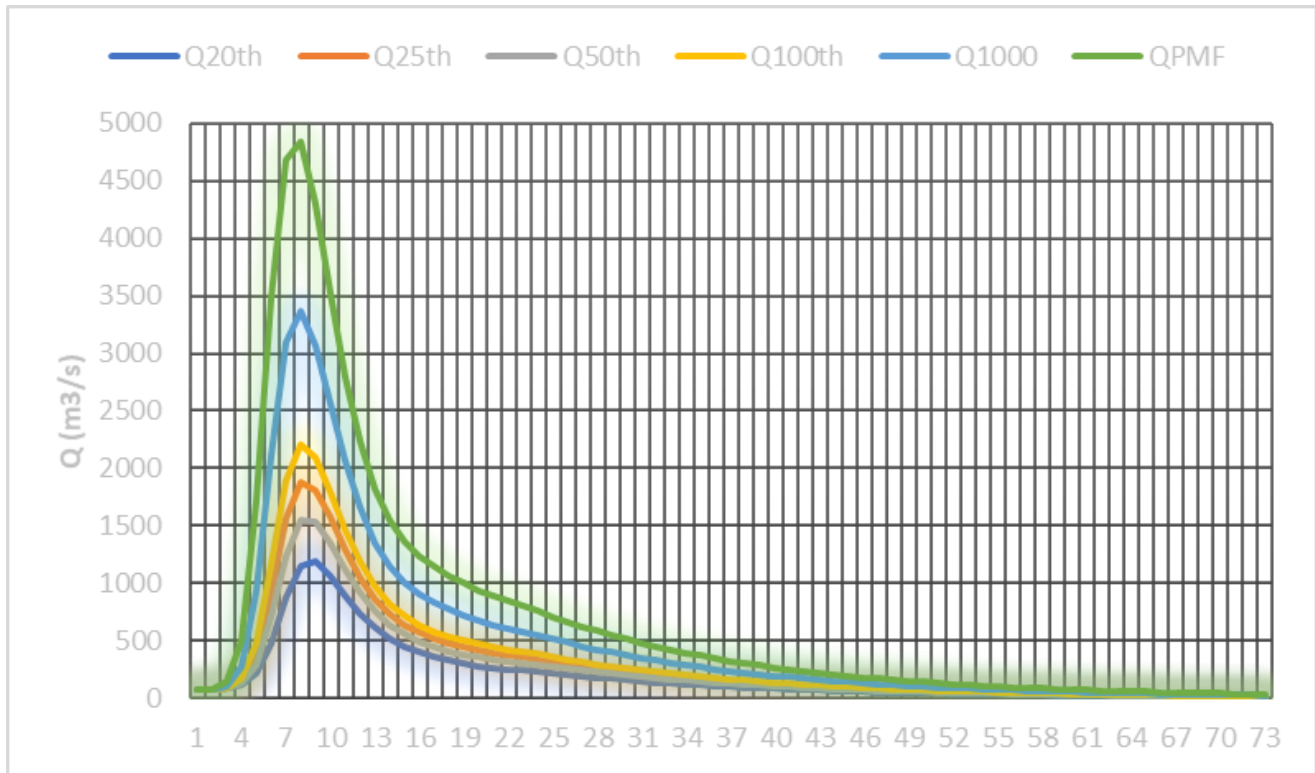


Fig. 7 Design flood hydrograph using the SCS-CN method

4.2. Flood Routing

The basic principle of routing on the dam is developed from the following continuity equation:

$$I - O = \frac{d_s}{dt}$$

where:

I - inflow to the reservoir (m^3/s);

O - outflow through the spillway (m^3/s);

$\frac{d_s}{dt}$ - storage change within time (hours).

$$\left(\frac{I_1 + I_2}{2} \right) \Delta t + \left(\frac{O_1 + O_2}{2} \right) \Delta t = S_2 - S_1$$

where:

S_1 - reservoir storage at the beginning of time t ;

S_2 - reservoir storage in time t ;

I_1 - inflow to the reservoir at the beginning of time t ;

I_2 - inflow to the reservoir in time t ;

O_1 - outflow through the spillway at the beginning of time t ;

O_2 - outflow through the spillway in time t .

The condition of Limboto Lake currently has two outlets: with and without a gate, as presented in the Figure below.



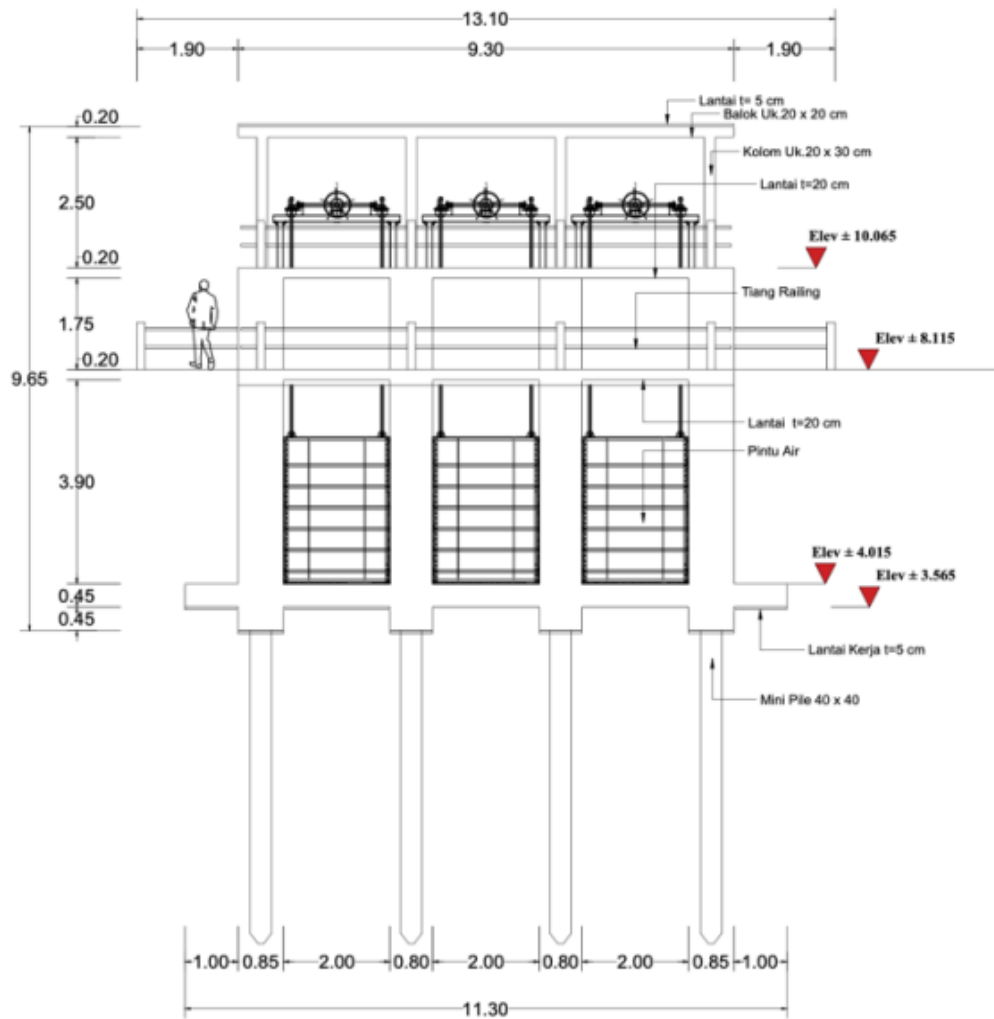


Fig. 8 Gate outlets in the Limboto Lake catchment area

The scenario of flood operation when opening-closing the spillway gate in Limboto Lake is as follows:

a. The initial storage condition at a 4-m elevation with a closed spillway gate;

b. The initial storage condition at a 4-m elevation with a 4-m opened spillway gate.

Table 16 presents the recapitulation of flood routing in the Limboto Lake storage.

Table 16 Recapitulation of the Flood Routing in the Limboto Lake storage

No	Return period	Inflow (m ³ /s)	Scenario			
			Initial Elv 4m, closed gate		Initial Elv 4m, opened gate: 4m	
			Outflow (m ³ /s)	Elv MAB	Outflow (m ³ /s)	Elv MAB
1	Q 25 years	1553.2	73.5	5.5	98.3	5.5
2	Q 50 years	1876.7	81.7	5.8	111.5	5.7
3	Q 100 years	2205.8	90.0	6.0	125.1	5.9
4	Q 1000 years	3364.0	117.6	6.6	172.6	6.5
5	Q PMF	4839.4	150.3	7.3	228.9	7.2

5. Conclusion

This research intends to evaluate the condition of Limboto Lake as a flood control storage in Gorontalo Province. The decrease in the lake storage power causes Limboto Lake to experience floods in almost

every rainy season. Besides being an income source for fishermen and a tourism facility, Limboto Lake also plays a role in flood control in Gorontalo Province. The storage of Limboto Lake is the retention of flooding of 23 affluents before being released into Bolango River

that has the estuary in Bone River.

Based on the analysis result of flood and storage capacity in Limboto Lake, it can be concluded as follows:

1. The rainfall data availability, which is from six rainfall stations spread in the Limboto Lake catchment area, is still less (less than 20 years); therefore, to complete and generate the rainfall data, the result of satellite observation, Global Precipitation Measurement (GPM) with the spatial resolution $0.10^\circ \times 0.10^\circ$, is used from January 1, 2001, to December 31, 2022;

2. The design flood due to some return periods are $1,553.2 \text{ m}^3/\text{dt}$ for $Q_{25\text{years}}$; $1,876.7 \text{ m}^3/\text{s}$ for $Q_{50\text{years}}$; $2,205.8 \text{ m}^3/\text{s}$ for $Q_{100\text{years}}$; $3,364 \text{ m}^3/\text{s}$ for $Q_{1000\text{years}}$; and $4,839.4 \text{ m}^3/\text{s}$ for Q_{PMF} ;

3. The result of flood routing in Limboto Lake is obtained that the condition of embankment is still safe if there is happened the design flood less than 1,000 years return period. However, if there the PMF discharge, it will lead to Limboto Lake embankment overtopping.

The result of the optimization scenario is that the flood storage capacity of Hongze Lake increased from 12.5 m to 13.0 m, and that of Luoma Lake - from 22.5 m to 23.5 m. The increase can also be used for fulfilling the water need in the downstream, which is 413 million m^3 .

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