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Evaluation of Flood Pump Capacity in the Madiun City Area to Overcome Waterlogging in the Catchment Area of the City's Drainage Channel

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ABSTRACT

Flooding has become an increasingly critical global challenge, with urban areas experiencing more frequent and severe flood events due to climate change and rapid urbanization. This research evaluates the capacity of flood pumps in Madiun City, Indonesia, to address waterlogging issues in urban drainage catchment areas. The objective of this study is to assess the adequacy of existing pump systems using hydrological analysis and determine the required pump capacity improvements for effective flood control. This research employed a quantitative descriptive method, utilizing primary data from field measurements and secondary data from relevant agencies. The study area encompasses eight pump houses in Madiun City with varying pump capacities. Hydrological analysis was conducted using rainfall data from 2011 to 2021, applying the Nakayasu Synthetic Unit Hydrograph method and Pearson III Log distribution for design rainfall calculation. Results indicate that existing drainage channels can accommodate flood discharge for return periods up to 25 years but require normalization for 50-, 100-, and 1000-year return periods. The Cassowary pump house analysis showed that the current pump capacity of 3 m³/s with a 4.23 m head is adequate for the 10-year return period. The research demonstrates that pump system effectiveness is crucial for flood control in low-lying urban areas. This study contributes to flood management literature by providing a systematic evaluation framework for urban pump systems and offers practical recommendations for municipal infrastructure planning.

KEYWORDS *Drainage System, Flood Control, Hydrological Analysis, Nakayasu Method, Pump Capacity, Urban Flooding, Waterlogging*

INTRODUCTION

Madiun is a very strategic transit city because it has a flat land topography that makes it easy for buses and trains to cross while supporting inland areas known for their cultural and tourism potential. The soil in Madiun has a fairly good structure, allowing it to absorb rainwater efficiently. This condition causes groundwater discharge to be almost constant throughout the year, making it easier for residents to obtain a clean water supply to meet their daily needs (Indartini et al., 2020; Newman et al., 2016; Ramirez-Guerrero et al., 2022). Shallow water sources are usually located at a depth of 8 meters, while artesian well water can be found at a depth of 90 meters. Madiun City is located in the Bengawan Solo watershed. A large river known as the Madiun River flows west of the city, dividing it into two parts. About 82% of the city's area is east of the river, while 18% is west. The Madiun River has two tributaries, namely the Sono River and the Catur River.

Madiun has long struggled with major flooding problems. Since ancient times, the Madiun River, also known as Bengawan Madiun, has often caused flooding in its basin (Amelia & Husain, 2022). Every year, seasonal floods due to the overflow of Bengawan Madiun submerge rice fields and houses of residents living on the riverbanks, including several sub-districts in Ponorogo Regency upstream and downstream of Ngawi Regency (Stekom, 2023).

This excess water raises the water level of Bengawan Solo, resulting in repeated floods in the Bojonegoro Regency area. With its geography located in the lowlands, Madiun City is vulnerable to waterlogging when there is high rainfall. This phenomenon is a serious threat to urban residents and existing urban infrastructure (Fitriyah et al., 2019).

The flood problem in Madiun City is further exacerbated by complex drainage patterns. Madiun City has two types of urban drainage systems, namely macro drainage and micro drainage, consisting of a total of 32 channels. However, these drainage channels are often insufficient to accommodate a very large volume (discharge) of water, causing frequent waterlogging along these channels (Prawati & Fajri, 2021). Additionally, siltation, narrowing of river bodies, sedimentation, and inadequate or poorly maintained drainage systems also contribute to this problem (Hetwisari et al., 2022; Lindawati et al., 2021). In such situations, an efficient water management system is crucial to reducing the adverse effects of flooding.

An efficient water management system includes planning and maintaining infrastructure to optimally manage water flow, such as retention ponds, clean and well-maintained drainage channels, and weather monitoring technology (Hijah et al., 2023; Ihsan et al., 2022; Mildawani, 2024; Sukerte, 2022; Zeffitni, 2016). This system also involves community participation in maintaining the cleanliness of waterways (Kurnianingsih et al., 2021). An efficient water management system reduces the adverse effects of flooding by increasing drainage capacity and effectiveness, reducing water load in channels, and preventing blockages. This results in reduced frequency and intensity of flooding, minimizing damage and losses (Sohn et al., 2020).

On the Madiun River section crossing Madiun City, an embankment has been built to prevent floods from overflowing and inundating the city. Due to the height of this embankment, rainwater flow from city drainage cannot directly flow into the Madiun River by gravity. Therefore, drainage in Madiun City must be assisted by water pumps. The drainage system uses two types of systems: the gravity system and the pumping system. These pumps are quite effective as flood control measures (Zulbihar & Sedyowati, 2022). There are a total of 8 pump houses functioning to carry out flood control. These are located as follows: Patihan pump house has 2 pumps, Pandan pump house has 6 pumps, Pancasila pump house has 4 pumps, Beteng pump house has 2 pumps, Madiun Lor pump house has 2 pumps, Taman pump house has 1 pump, Kaswari pump house has 3 pumps, and Sogetan pump house has 1 pump. Of these, 7 pump houses are in good condition, with only the Garden pump house suffering minor damage.

Of the several flood pumps built in Madiun City, the existing pump capacity is still insufficient to drain the city’s drainage water into the Madiun River, resulting in inundation/flooding in the city. The main purpose of these flood pumps is to evacuate water from drainage channels to safer areas, such as rivers or infiltration zones. However, the effectiveness of this flood pump system requires careful evaluation.

Evaluation of flood pump capacity in the Madiun City area to overcome waterlogging in the catchment area of the city's drainage channel is very important. The goal is to ensure that this system can still cope with waterlogging occurring in the catchment area of the city's drainage canals. Without proper evaluation, the capacity of flood pump systems may become inadequate in the face of increasing flooding challenges. As the intensity of extreme rainfall increases due to climate change, evaluating the capacity of pumps in Madiun is increasingly urgent. Without system improvements, the risk of socio-economic losses and damage to urban infrastructure will be greater.

The novelty of this research lies in its integrated application of the Nakayasu Synthetic Unit Hydrograph method specifically for urban pump capacity evaluation in low-lying areas, which has not been comprehensively studied in previous research. Unlike earlier studies that primarily focused on drainage design or isolated pump assessments, this research provides a systematic framework for evaluating pump effectiveness using advanced hydrological modeling.

In this study, the problem formulation includes several key questions aimed at understanding rainwater management and inundation in the Madiun City area, including the calculation of maximum design rainfall and discharge generated from rainfall in each catchment area, determination of Synthetic Unit Hydrograph (SUH) using the Nakayasu method, and pump capacity and storage volume requirements. This research aims to: (1) conduct a comprehensive evaluation of flood pump capacity in Madiun City using advanced hydrological analysis, (2) determine adequate pump specifications for effective flood control across different return periods, (3) provide technical recommendations for pump system optimization, and (4) contribute to urban flood management knowledge through systematic pump capacity assessment methodology. The benefits of this research include academic contributions to flood control engineering literature, practical guidelines for municipal infrastructure planning, a technical framework for pump system evaluation, and policy recommendations for sustainable urban drainage management. The study limits are set to ensure clear focus, covering only the Madiun City area and inundation occurring in the city's drainage channels, prioritizing flood pump capacity evaluation without addressing other aspects such as maintenance and overall system efficiency. The authenticity of this study lies in its specific approach to evaluating flood pump capacity in a unique geographic context, as well as its new contribution to understanding the effectiveness of such systems by delving deeper into relevant flood discharge calculations for each catchment area. This research is expected to provide useful insights for the development of flood management infrastructure in Madiun City and aid future planning.

METHOD

This research was conducted in Madiun City, East Java, Indonesia, specifically focusing on the urban drainage catchment areas and pump house locations distributed across the city. The study utilized a quantitative descriptive research approach with explanatory design to systematically evaluate pump capacity effectiveness. Primary data collection included field measurements of water levels, pump specifications, and drainage pipe dimensions. Secondary data were obtained from the Irrigation Public Works Office in Madiun, including 11 years (2011-2021) of rainfall data from four rain stations: Madiun, PG Rejoagung, Kanigoro, and Klegen stations. Data analysis techniques comprised hydrological analysis using statistical methods for rainfall frequency distribution, synthetic unit hydrograph calculation using the Nakayasu method, and pump technical analysis including efficiency and power calculations.

In conducting this research, data was processed using descriptive and quantitative methods, where this approach involves collecting primary data from the field and secondary data from related agencies, as well as referring to relevant literature. Data analysis was carried out in accordance with the previously discussed theory, and final conclusions were reached regarding the drainage pump capacity required to overcome waterlogging in the catchment area of the city's drainage canal area with a 10-year re-planning period. This research begins with the identification of the problem which aims to find the source of the problem with careful observation and not rushing in drawing conclusions. Once the source of the problem is identified, a literature review is conducted to study relevant previous research, so as to determine the next steps and utilize existing methods as comparative data. Data collection consists of primary data collected independently, including water level and length of drainage pump pipes, as well as secondary data obtained from related agencies, including rainfall, drainage network maps, pump capacity, and pump power. Furthermore, data analysis is carried out through several stages, starting from hydrological analysis to calculate regional average rainfall and design rainfall using various methods as a comparison, then continued with surface runoff analysis to predict flood discharge, and ending with pump technical analysis to calculate motor power and pump efficiency based on data obtained from the factory. Through these stages, the research is expected to provide a comprehensive picture of the drainage pump capacity needed in the area studied.

RESULTS AND DISCUSSION

Hydrological Data Quality Analysis

1. Hydrological Data

Hydrological analysis is the initial stage of a study on flood control. This study will use annual maximum rainfall data for 11 years (2011-2021) from 4 rain stations, namely: Madiun Rain Station, PG Rejoagung Rain Station, Kanigoro Rain Station and Klegen Rain Station.

Table 1. Rainfall Data

Year	Madiun	PG. Rejoagung	Kanigoro	Klegen	Average
2011	144	110	85	100	109,75
2012	100	83	93	114	97,50
2013	158	82	135	109	121,00
2014	60	73	75	62	67,50
2015	128	104	113	125	117,50
2016	125	171	180	120	149,00
2017	153	150	75	155	133,25
2018	85	79	63	82	77,25
2019	95	92	113	94	98,50
2020	122	114	80	75	97,75
2021	114	97	82	65	89,50

Source : Irrigation Public Works Office in Madiun

Before hydrological data is used for advanced hydrological analysis, it must go through several tests to ensure that the data can be used. One of them is a consistency test using the RAPS method, followed by data abnormalities and finally a persistence test.

Data Consistency Test

The method used for data consistency testing is the RAPS (*Rescaled Adjusted Partial Sums*) method. This method is a test with the cumulative deviation of the average value divided by the cumulative root of the average deviation of the square to the average value, to get the statistical value then compared to the permissible conditions, if it is smaller then the data is still within the consistent limit. The following are the results of the rain data consistency test with the RAPS method.

Table 2. Data Consistency Test

No	Year	Rainfall	Yi - Yrerata	Cs*	D2	Cs**	[Sk**]
1	2011	109,75	4,43	4,43	1,79	0,19	0,19
2	2012	97,50	-7,82	-3,39	5,56	-0,15	0,15
3	2013	121,00	15,68	7,86	22,36	0,34	0,34
4	2014	67,50	-37,82	-22,14	130,02	-0,97	0,97
5	2015	117,50	12,18	-25,64	13,49	-1,12	1,12
6	2016	149,00	43,68	55,86	173,46	2,45	2,45
7	2017	133,25	27,93	71,61	70,93	3,14	3,14
8	2018	77,25	-28,07	-0,14	71,62	-0,01	0,01
9	2019	98,50	-6,82	-34,89	4,23	-1,53	1,53
10	2020	97,75	-7,57	-14,39	5,21	-0,63	0,63
11	2021	89,50	-15,82	-23,39	22,75	-1,02	1,02

No	Year	Rainfall	Yi - Yrerata	Cs*	D2	Cs**	[Sk**]
Sum		1.158,50			521,40		
Rerata		105,32					
n		11					
Sk** max.		3,14					
Sk** min.		-1,53					

Source : Calculation Results, 2024

Q = 3,14
R = 4,66
Q/n0.5 count = 0,53
Q/n0.5 table = 1,06
Conclusion = Accepted
R/n0.5 count = 0,65
R/n0.5 table = 1,22
Conclusion = Accepted

For 90% Consistency Limit
Q/n0.5 = 0,53 < 1.06 → Qualify
R/n0,5 = 0,65 < 1.22 → Qualify

From the analysis carried out with a degree of confidence, 90% of the statistical value obtained was smaller than the permissible limit, so it can be interpreted that the rain data in the study area is consistent and can be used for further analysis.

Data Anomaly Test (Outlier)

The data that has been consistent is then tested again with a data abnormality test. This test is used to determine whether the maximum and minimum data from the data set that has been tested is suitable for use or not. The test used is the inlier-outlier test, where data that deviates from the two threshold limits, namely the lower threshold (Xi) and the upper threshold (Xh) will be eliminated. The following are the results of the inlier-outlier test for rain data.

Table 3. Inlier test – outlier rain data

No	Year	Σ Monthly Rainfall	log x	Information
1	2011	109,75	2,040	Upper threshold value, Xh
2	2012	97,50	1,989	Xh = 200,620
3	2013	121,00	2,083	
4	2014	67,50	1,829	Lower threshold value, Xi
5	2015	117,50	2,070	Xi = 52,692
6	2016	149,00	2,173	
7	2017	133,25	2,125	Then all data can be used
8	2018	77,25	1,888	
9	2019	98,50	1,993	
10	2020	97,75	1,990	
11	2021	89,50	1,952	
	n	=	11	
	Stdev.	=	0,101	
	Mean	=	2,01	
	Kn	=	2,88	

Source: Calculation Results, 2024

From the results of the inlier-outlier test examination of the rain data, it is known that the rain data is still in the range of the upper and lower thresholds of the analysis results. So it can be concluded that the rain data is data used.

Table 4. Recapitulation of Inlier-Outlier Test of Rain Data

Rain Max	Min Rainfall	Upper threshold	Lower threshold	Information
149,00	67,50	200,62	52,69	Worth Using

Source: Calculation Results, 2024

Trend Absence Test

Trend analysis is used to determine whether or not there are changes in hydrological variables due to human influence or natural factors. In this study, the method used to test the absence of trends in the periodic series is the spearman method, because the spearman method can work for only one type of hydrological variable, where in this case it is the annual rainfall or maximum rainfall. The following are the results of the analysis of the absence of trend test from the available rain data.

Table 4. Test the Absence of Trend with Spearman Method Rank Correlation

No.	Year	Σ Annual Rainfall	Tt Rating	Rt Rating	German	Dt2
1	2011	109,75	1	5	-4	16
2	2012	97,50	2	8	-6	36
3	2013	121,00	3	3	0	0
4	2014	67,50	4	11	-7	49
5	2015	117,50	5	4	1	1
6	2016	149,00	6	1	5	25
7	2017	133,25	7	2	5	25
8	2018	77,25	8	10	-2	4
9	2019	98,50	9	6	3	9
10	2020	97,75	10	7	3	9
11	2021	89,50	11	9	2	4
					Sum	178
		n		11		
		KP		0,1909		
		Stuttgart		0,5835		
		dk		9		
		tc		2,262		
		tc > thitung		Accepted		

Source : Calculation Results, 2024

Based on the results of the analysis carried out, the calculated t value obtained was 0.5835, which is smaller than the critical t value allowed for 11 data, which is 2.262. So it can be concluded that the rain data used does not show a trend so it can be used for analysis.

Table 5. Recapitulation of Trend Absence with Ranking Correlation Ranking Method

n	KP	Stuttgart	dk	tc	Information
11,00	0,19	0,58	9,00	2,26	Accepted

Source : Calculation Results, 2024

Stationary Test

After testing the absence of trends, if the periodic series does not show a feeding trend, a follow-up analysis is carried out, namely a stationary test. If there is a trend, then the periodic series can be analyzed for the resulting trend line. Stationary tests can be carried out by conducting variant stability tests and average stability tests from rain data, as well as stationary test results for average rainfall data at the study site.

Table 6. Rain Data Stationary Test

Group I			Group II		
No.	**Year**	**Σ Annual Rainfall**	**No.**	**Year**	**Σ Annual Rainfall**
1	2011	109,75	7	2017	133,25
2	2012	97,50	8	2018	77,25
3	2013	121,00	9	2019	98,50
4	2014	67,50	10	2020	97,75
5	2015	117,50	11	2021	89,50
6	2016	149,00			
VARIANT STABILITY TEST (F TEST)					
n1	6		n2	5	
Rerata	110,38		Rerata	99,25	
Sd	27,06		Sd	20,84	
dk1			5		
dk2			4		
Calculation			2,527		
Fcr α 5 %			4,950		
Fcount < Fcr,			Accepted		
AVERAGE STABILITY TEST (T TEST)					
n1	6		n2	5	
Rerata	110,38		Rerata	99,25	
Sd	27,06		Sd	20,84	
dk			9		
s			27,007		
Stuttgart			0,680		
TCR A 5 %			2,262		
thitung < tcr,			Accepted		

Source : Calculation Results, 2024

The results of the analysis of the variance stability test (F Test) and the average stability test (T Test) of the rain data at the study site were accepted because the tcount and Fcount values were smaller than the ttable and Ftabel for α 5% or 95% confidence so that the rain data could be used for further analysis. The following are the results of the stationary test at the rain station.

Table 7. Recapitulation of Rain Data Stationary Test

Calculation	Stuttgart	Fcritical	tcritical	Information
2,53	0,68	4,95	2,26	Accepted

Source : Calculation Results, 2024

Persistence Test

Persistence is the non-dependence of any value in a periodic series. To carry out persistence testing, the magnitude of the serial correlation coefficient must be calculated. One method to determine the serial correlation coefficient is the spearman method. The following are the results of the rain data persistence test.

Table 8. Rain Data Persistence Test

No.	Year	Σ Annual Rainfall	Rt Rating	of	of2
1	2011	109,8	5	-	
2	2012	97,5	8	-3	9
3	2013	121,0	3	5	25
4	2014	67,5	11	-8	64
5	2015	117,5	4	7	49
6	2016	149,0	1	3	9
7	2017	133,3	2	-1	1
8	2018	77,3	10	-8	64
9	2019	98,5	6	4	16
10	2020	97,8	7	-1	1
11	2021	89,5	9	-2	4
				Sum	242

m	10
KS	-0,467
Stuttgart	-1,492
dk	8
tc	2,306
tc > tcount	Accepted

Source : Calculation Results, 2024

Table 9. Rain Data Persistence Test Recapitulation

KS	Stuttgart	tcritical	Information
-0,467	-1,492	2,306	Accepted

Source : Calculation Results, 2024

Rainfall Quality Analysis

After the hydrological data is tested for quality and it is ensured that the data can be used for further analysis. Then a frequency distribution analysis will be carried out to find the design rainfall by previously calculating the average rainfall. In accordance with the provisions of the Ministry of PUPR, the repetition period used is the repetition period of 2,5,10,20,25,50,100 and 1000 years. The planned rainfall must be distributed matching test to ensure that *the frequency distribution* represents *the probability frequency* or not.

Regional Average Rainfall Analysis

Analysis of regional average rainfall is used to obtain a value of rainfall that can represent the entire value in the Madiun City watershed. This value will also be used for validation analysis. The method used in this analysis is the Thiessen Polygon Method using the help of ArcGIS 10.8 software. In this study, there are 4 rain station posts located around the Madiun City watershed, so the average rainfall value of the region is needed.

a) Calculation of the Value of the Influence Area Factor of the Rain Station (Kr)

From the analysis using ArcGIS 10.8, the area of each rain station was obtained as follows:

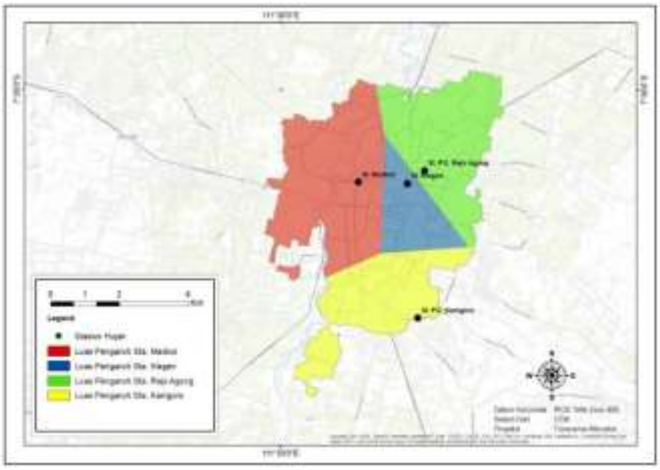

Figure 1. Thiessen Polygon and Rain Station Distribution

Source: Depiction Results, 2024

Table 10. Extent of Influence of Rain Stations (Kr) using ArcGIS 10.8

No	Rain Station	Area (km2)	Kr
1	St. Madiun	12,195	0,37
2	Rejoagung	7,776	0,23
3	Klegen	4,017	0,12
4	Kanigoro	9,241	0,28
		33,23	1

Source : Calculation Results, 2024

Based on Table 11, the Kr value is obtained at each rain station post. The value of this Kr is influenced by the area of influence of each rain station post on the area of the Madiun City watershed and the overall area of the Madiun City watershed. The results of this Kr calculation are then used to calculate the average rainfall of the area using the Thiesen Polygon Method. The example of calculation using rainfall data in 2011 is as follows:

$$
\begin{aligned}
R &= (X_1.\ Kr_1) + (X2.Kr2) + (X3.Kr3) + (X4.Kr4) \\
&= (144.0,37) + (110.0,23) + (85.0,28) + (100.0,12) \\
&= 114,32 \text{ mm}
\end{aligned}
$$

Table 11. Average Rainfall Thiessen

Year	Madiun City	PG. Rejoagung	Kanigoro	Klegen	Thiesen Average Rainfall
2011	144	110	85	100	114,32
2012	100	83	93	114	95,77
2013	158	82	135	109	127,90
2014	60	73	75	62	67,46
2015	128	104	113	125	117,85

Year	Madiun City	PG. Rejoagung	Kanigoro	Klegen	Thiesen Average Rainfall
2016	125	171	180	120	150,46
2017	153	150	75	155	130,85
2018	85	79	63	82	77,12
2019	95	92	113	94	99,18
2020	122	114	80	75	102,77
2021	114	97	82	65	95,20

Source : Calculation Results, 2024

Distribution Selection

Before rainfall data is carried out, a frequency distribution analysis must be carried out first so that it can be used for future calculation analysis. The following are the results of the distribution selection calculation:

Table 12. Distribution Selection Calculation Results

No	Year	Rainfall	(x-xrt)^3	(x-xrt)^4
1	2011	114,32	365,27	2611,04
2	2012	95,77	-1481,83	16893,90
3	2013	127,90	8904,45	184562,19
4	2014	67,46	-62631,33	2487265,59
5	2015	117,85	1218,62	13016,47
6	2016	150,46	81111,93	3511125,21
7	2017	130,85	13277,80	314413,74
8	2018	77,12	-27143,89	815760,72
9	2019	99,18	-509,20	4066,11
10	2020	102,77	-85,32	375,60
11	2021	95,20	-1714,73	20523,97
	Sum	1178,85	11311,78	7370614,55
	Rerata	107,17	n	11
	Sd	24,22	Cs	0,10
	Ck	3,60		

Source : Calculation Results, 2024

Data distribution testing requirements for the use of frequency distribution analysis

Table 13. Data Distribution Testing Requirements

Normal Distribution	Gumbel Distribution	Pearson III Log Distribution
- 0.05 < Cs < 0.05	CS > 1.1395	
2.7 < Ck < 3.3	Ck > 5.4	
Source: Harto, Sri. 1993. Hydrological Analysis.		
- 0.05 < Cs < 0.05	0.0973 >1.1395	No Limits
Not meeting	Not meeting	
2.7 < Ck < 3.3	3.6 > 5.4	No Limits

Normal Distribution	Gumbel Distribution	Pearson III Log Distribution
Not meeting	Not meeting	

Source : Calculation Results, 2024

Frequency Distribution Analysis

The selection of this rainfall frequency distribution is to find out the type of distribution of existing rainfall data and the appropriate frequency distribution for the calculation of the rainfall design. In connection with the rainfall frequency distribution test, there are several types of frequency distributions that we commonly know, including: Gumbel, Normal, Pearson III Log and others. The frequency distribution analysis was carried out with *Hydrognomon* software with the results which can be seen in Table 15.

Table 14. Frequency Analysis Recapitulation Results *Software* Hydrognomon

Distribution Method	Re-Period								Distribution Conformity Test	
	2 Years	5 Years	10 Years	20 Years	25 Years	50Years	100 Years	1000 Years	Chi Square 5%	Kolmogorov-Smirnov 5%
Normal	107,171	127,555	138,21	147,009	149,572	156,912	163,515	182,016	Accepted	Accepted
Gumbel	103,191	124,603	138,78	152,378	156,692	169,98	183,17	226,755	Accepted	Accepted
Log Pearson III	102,893	126,619	142,492	157,894	162,829	178,229	193,871	249,008	Accepted	Accepted

Source: Calculation Results, 2024

Distribution Compatibility Test

This conformity test is checked to find out the truth of the frequency distribution hypothesis. With this test examination, several things will be known such as:

1) The truth between the observation results and the expected or theoretically obtained distribution model.
2) The truth of the hypothesis is accepted or rejected.

The hypothesis of an initial design is a temporary formulation of something that is made and requires an investigation into the suitability of the distribution made. Based on Table 15, it is known that the average frequency distribution conformity test used shows results that are considered appropriate at a degree of confidence of 5%. The distribution chosen for this study was the Pearson III Log distribution.

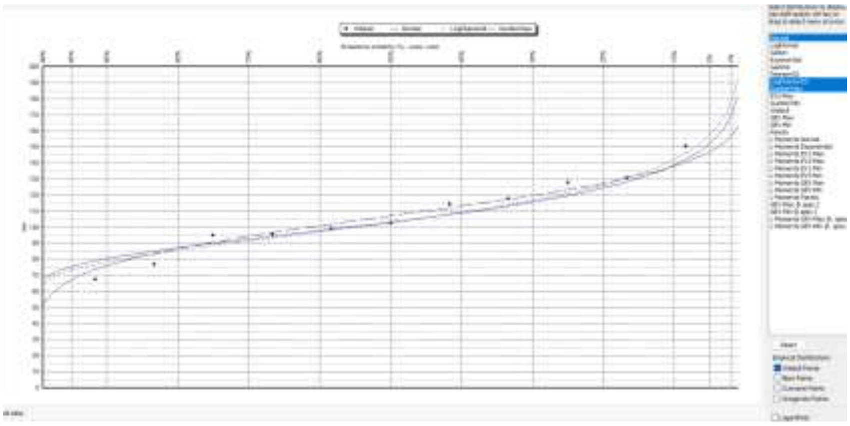

Figure 2. Plotting Yield Frequency Distribution *Running* Hydrognomon

Analysis of Flood Control Building Handling

As the result of the large calculation of the flood discharge design with the Rational Method and the Nakayasu Method in each watershed, namely the Cassowary Watershed, Pancasila Watershed, Beteng Watershed, Madiun Lor Watershed, Patihan Watershed, and Sogaten Watershed.

1. Cassowary Channel Dimension Discharge Recapitulation

From the results of the flood discharge calculation for the calculation of flood pump capacity, one of the flood pump stations in the Madiun City area was taken, namely the Cassowary Flood Pump Station, in the flood management sub-system at the Cassowary Pump Station 3 Flood Pumps are installed using an intake pond system and one drainage channel outlet that leads to the Madiun River.

Table 16. Results of Cassowary Channel Dimension Discharge Recapitulation Calculation

No	Period Birthday (Birthday)	Channel Segments	Capacity Channels	Intensity Rain(mm)/ hour	Debit Flood (m³/s)	Ket Dimension Channels	Solution
1.	2	Kasuari	30.878,55	42,786	18,599	Enough	-
2.	5	Kasuari	30.878,55	52,615	22,839	Enough	-
3.	10	Kasuari	30.878,55	59,211	25,702	Enough	-
4.	20	Kasuari	30.878,55	65,611	28,480	Enough	-
5.	25	Kasuari	30.878,55	67,662	29,370	Enough	-
6.	50	Kasuari	30.878,55	74,061	32,148	No Sufficient	Normalization
7.	100	Kasuari	30.878,55	80,561	34,969	No Sufficient	Normalization
8	1000	Kasuari	30.878,55	103,473	44,914	No Sufficient	Normalization

The existing discharge capacity of the Cassowary city drainage channel in Figure 3 is only able to accommodate 30,878.55 m^3/sec, in the results of the calculation of the discharge of the cassowary channel during the 50 years, 100 years, and 1000 years for the channel dimensions and water discharge capacity in the Cassowary city drainage channel are insufficient, so it is necessary to carry out normalization and engineering.

Figure 3. Existing Dimensions of Cassowary City Drainage Channels

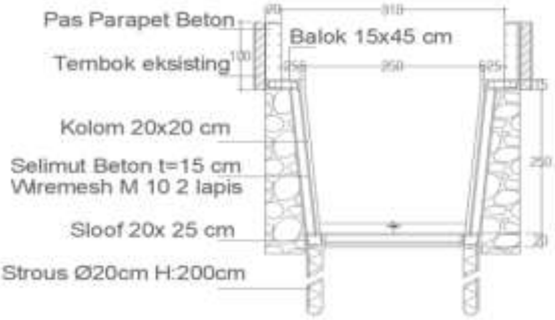

Figure 4. Recommended dimensions of Cassowary city drainage channels

In Figure 4 The dimensions of the drainage channel of the Kasuwari city are suggested where the calculation of channel data is presented:

Rectangular Channel

- $b = 3.1$ m
- $h = 1$ m
- $n = 0.015$ (ordinary concrete channels)
- $s = 0.0075$

Calculate wet cross-section area

$A = b.h = 3.1 \times 1.0 = 3.1$ m^2

$P = b+2h = 3.1+2.0\,(1) = 5.1$ m

Calculate hydraulic radius

$R = A/P = 3.1/5.1 = 0.61$ m

$Qs = A \times (1/n) \times R(2/3) \times S(1/2)$
$= 3.1 \times (1/0.015) \times (0.61)\ 2/3\ .\ (0.0075)\ 1/2$
$= 3.1 \times 66.666\ 7 \times 0.7131 \times 0.0866$
$= 12.7$ m^3/s

- $b = 2.5$ m
- $h = 2.5$ m
- $n = 0.015$ (ordinary concrete channels)
- $s = 0.0075$

Calculate wet cross-section area

$A = b.h = 2.5 \times 2.5 = 6.25\ m^2$
$P = b+2h = 2.5+2.5\ (1) = 5\ m$
Calculate hydraulic radius
$R\ = A/P = 6{,}25/5 = 1.25\ m$
$Qs = A \times (1/n) \times R(2/3) \times S(1/2)$
$= 6.25 \times (1/0.015)\times(1.25)\ 2/3\ .\ (0.0075)\ 1/2$
$= 6.25\times\ 66.666\ 7 \times 1.1696 \times 0.0866 = 42.175\ m^3/sec$
Total $Qs = 12.7\ m^3/sec + 42.175\ m^3/sec = 54.875$

Table 17. Results of the Recommended Cassowary Channel Dimension Discharge Recapitulation Calculation

No	Birthday	Channel Segments	Capacity Channels	Intensity Rain(mm)/ hour	Debit Flood (m^3/s)	Dimension Channels Information	Solution
1.	2	Kasuari	30.878,55	42,786	18,599	Enough	-
2.	5	Kasuari	30.878,55	52,615	22,839	Enough	-
3.	10	Kasuari	30.878,55	59,211	25,702	Enough	-
4.	20	Kasuari	30.878,55	65,611	28,480	Enough	-
5.	25	Kasuari	30.878,55	67,662	29,370	Enough	-
6.	50	Kasuari	54.875	74,061	32,148	Enough	-
7.	100	Kasuari	54.875	80,561	34,969	Enough	-
8	1000	Kasuari	54.875	103,473	44,914	Enough	-

From the engineering of the existing cassowary city drainage channel by adding a pair of concrete parapets as high as 1 meter and normalizing soil sediment and adding a depth of channel elevation to 50 cm deep, it has increased the width of the wet cross-section of the channel and increased the capacity of the channel water discharge as the attached calculation is made. So it can be concluded that by engineering the drainage channels of the existing cassowary city, it can overcome the water reservoir discharge of the cassowary channel in the recalculation of 50 years, 100 years and 1000 tofu

A. Pump Technical Analysis

Calculated based on the technical analysis of the 2-year pump (Q2) at the Cassowary Flood Pump House:

Discharge Flood Plan
$Q \quad = 0.278 \times L \times I \times H$
$= 0{,}278 \times 0{,}74 \times 42{,}756 \times 2{,}11$
$= 18{,}559\ m3/sec$
Head Pump

Discharge Pump in the Cassowary Pump House 3 pieces with 1 pump capacity 1000 ltr/s = 1 m^3/sec. The diameter of the pipe can be determined by the following equation:

D =

Where V pipe = 1.5 m/s (specified) 4Q
So: √ πV
D = 4 x 3 m³/s
wx 1.5 m³/sec
= 1.6 m = 63 in
= Taken pipe diameter of 63 in

Head loss

HLT is a head loss that occurs in the pump, which includes major head loss caused by friction on the pipe and minor head loss caused by piping fittings that occur in the suction and discharge parts of the pump Head loss mayor

To find out the magnitude of the major head loss that occurred, the following data is needed:

a. Panjang pipa discharge (L *discharge*
= 4510 + 750 + 450 +5600 + 675 (m) = 11985= 12 m

b. Pipe Material
Galvanize iron (e=0.15 mm=0,00015 m)

c. The speed of the water flow in the pipe

$$\overline{\mathrm{V}} = \frac{\mathrm{Q}}{\mathrm{A}} = \frac{Q}{\frac{\pi}{4} \times d^2} = \frac{3m^3/s}{\frac{3{,}14}{4} \times (1{,}6002m)^2} = 1{,}493\ m/s$$

Price $\frac{e}{d} = \frac{0{,}15 \times 10^{-3} m}{0{,}9m} = 1{,}1594 \times 10^{-4}$

d. Reynold number

$$\mathrm{Re} = \frac{\rho.\overline{V\,.d}}{\mu} = \frac{999kg/m^3 \times 1{,}493\ m/s \times 1{,}6002m}{1{,}14 \times 10^{-3}\ N.s/m^2} = 2{,}09 \times 10^6$$

From the moody diagram with the values = 0.000115794 and Re = 2093508 the value of the friction coefficient (*f*) was obtained as 0.013. $\frac{e}{d}$

The amount *of head loss* in *the suction* and *discharge* section is as follows:

$$hl = \frac{f\,\mathrm{L}\,\overline{V^2}}{\mu} = \frac{0{,}013 \times 12m\ (1{,}493m/s)^2}{2 \times 1{,}6002m \times 9{,}81m/s} = 0{,}0111\ m$$

The total *head loss of the major* is 0.0111 m

Head Loss Minor

Minor head loss occurs due to piping *fittings* in pump installations that include *suction* and *discharge parts*.

$$hl = (\mathrm{K}_{Lbow} + \mathrm{K}_{Lbow\ street} + \mathrm{K}_{entrance}) \times \frac{\overline{V^2}}{2g}$$

$$hl = \frac{(0.88 + 0{,}17 + 0{,}78)m \times (1{,}493\mathrm{m/s})^2}{2 \times 9{,}81\ m/s^2} = 0{,}2079\ m$$

Pump Head Calculation:

$$H = \left(\frac{P_2 - P_1}{\gamma}\right) + \left(\frac{{V_2}^2 - {V_1}^2}{2\mathrm{g}}\right) + (Z_2 - Z_1) + H_{LT}$$

Where:

$$P_2 = P_1$$

Hst = 450 + 750 + 4510 – 2300 + 500 = 3920 mm = 3.92 m

$V_1 = 0$

$$V_1 = \frac{3m^3/s}{\frac{1}{4} \times \pi \times 1{,}6002m^2} = 1{,}4917\, m/s$$

$$H = 0 + \left(\frac{(1{,}4917\, m^2/s) - 0^2}{2 \times 9{,}8\, m/s^2}\right) + 3{,}920 + 0{,}229 = 4{,}2251\, m$$

Water Power and Pump Power

The amount of *water horse power* is obtained by the following equation:

$$WHP = \rho \times g \times Q \times H$$

Where : = 999 Kg/m³ρ

g = 9.81 m/s²

so that:

$$WHP = 999 \frac{Kg}{m^3} \times 9.81 \frac{m}{s^2} \times 2 \frac{m^3}{s} \times 4{,}2251\, m$$

$$= 82813{,}57W \times \frac{1HP}{745{,}7\, W} = 111{,}05\, HP$$

Calculating the Overall Efficiency *of* the Pump (η_{op})

The *Overall* Pump Efficiency is obtained by the following equation:

$$A = \sqrt{\frac{Q_{SL} \cdot n}{1000}}$$

Where: Q_{SL} = discharge capacity *Impeller* Pump (liters/sec)

A = constant price

N = rotation *Impeller* Pump (rpm)

$$Q_{SL} = 3 \frac{m^3}{s} \times \left|\frac{1000\, liter}{1\, m^3}\right| = 3000\, liter/s$$

$$A = \sqrt{\frac{Q_{SL} \cdot n}{1000}}$$

$$A = \sqrt{\frac{3000 \frac{liter}{s} \times 720rpm}{1000}}$$

$$A = 46{,}47$$

Pump *overall* efficiency table (η_{op})

A	5	10	15	20	30	40	80
η_{op}	0.65	0.75	0.785	0.82	0.86	0.88	0.9

Based on the table above for price A = 8.96, the overall efficiency price *of* the pump (η_{op}) is = 0.76 = 76%. (price limit = 0.63 – 0.84)η_{op}

$$BHP = \frac{Q \times \gamma \times H}{\eta_{op}} = \frac{WHP}{\eta_{op}} = \frac{111{,}05\,HP}{0{,}76} = 146{,}12HP$$

BHP at *start* = 146.12 1.3 = 189.96×*HP*

Pump Specific Speed ()n_s

The specific speed of the pump can be calculated with the following equation:

$$n_s = 3{,}65n\frac{\sqrt{Q}}{H^{\frac{3}{4}}}$$

Where: H = 4.2251 m
Q = 3 m^3/s
n = 720 rpm

so:

$$n_s = 3{,}65 \times 720 \times \frac{\sqrt{3\,m^3/s}}{4{,}2251^{\frac{3}{4}}} = 1544{,}57\,rpm$$

After obtaining the value, the type of n_s*impeller* or the type of pump to be used is obtained, namely *the axial pump.*

CONCLUSION

This study concludes that Madiun City's existing drainage channels effectively handle flood discharges for return periods up to 25 years, but for larger events (50, 100, and 1000 years), channel normalization and structural upgrades like concrete parapets and elevation increases are necessary, as demonstrated in the Cassowary channel case. The Cassowary flood pump station, with an installed capacity of 3 m^3/s and a 4.23 m head using axial pumps, is adequate for managing 10-year flood events. This research highlights the essential role of integrated pump systems in urban flood control for low-lying areas and provides a validated assessment framework using the Nakayasu method. Future research should extend this evaluation to all eight pump houses in Madiun City to inform a comprehensive flood management plan, incorporate real-time monitoring for dynamic forecasting and pump operation, assess land use and climate change impacts on rainfall patterns, and explore integrating green infrastructure and sustainable urban drainage systems with existing grey infrastructure for more resilient and cost-effective flood mitigation.

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