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Analysis of Boezem Kalidami Pump Capacity To Cope With Floods East Surabaya Region

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Abstract

Kalidami river has a length of 4.2 kilometers with a stretch of channels from the Airlangga, Manyar, Kertajaya, to Kejawan Putih Tambak areas. Boezem Kalidami is located at the downstream of the Kalidami river which is also equipped with a pump house and a water gate. The Log Pearson III method produces a design rainfall of 127.38 mm / 24 hours so that the rainfall is very heavy. So that it will produce a maximum flood discharge of 40.3 m³ / s and when compared to the capacity of Boezem Kalidami which is only 49,440 m³, Boezem Kalidami will fill very quickly. Coupled with the total efficient pump capacity of only 15.33 m³ / s, Boezem is only able to accommodate the first 2 hours and an overflow of 1.58 meters from Boezem Kalidami's riverside at 2.14.

Keywords:
Boezem, Drainase, Flood, Kalidami, Pump Efficiency.

1. Introduction

1.1. Research Background

The general concept of drainage used in Indonesia is the conventional drainage concept, namely drainage "ditching area". This conventional drainage is an attempt to remove or drain excess water as quickly as possible to the nearest river. In this concept, rainwater that falls into an area must be discharged as quickly as possible into the river and then flows into the sea. This concept is used thoroughly in residential, rural, agricultural areas, and others. (Syarifudin, 2017)

The most effective way to maximize drainage is to widen and deepen the drainage channel. However, for a metropolitan city like Surabaya is not easy to conduct land acquisition for widening the river. So a possible solution is to maximize the performance of the drainage pump. It takes careful planning to the selection of appropriate drainage pump capacity can be maximized so that its performance with regard to the calculation of rainfall plans to return period of 10 years.

1.2. Problem Formulation

From the above description, due to the capacity requirements planning adequate drainage pumps, the issues to be discussed at the Final are as follows:

1) What is the maximum runoff discharge for the planned rainfall of 2, 5, and 10 years?
2) How big is the capacity of the existing boezem as temporary overflow storage?
3) How much capacity and effectiveness of the drainage pump needed to speed up water flow so that water does not overflow when compared to the current condition of the drainage pump?

2. Literature Review

According to Abduh, (2018) drainage or it can be referred to as ditching, can be interpreted as the process of disposing of water naturally or it can be man-made by humans from a place. Disposal methods also vary as to how to drain, drain, remove, and can also be also to divert the water.

Kodoatie & Sjarief, (2010) made a sequence of water flow in the drainage. This can be explained as follows:

1. Rain falls evenly on the residential area (settlement) R, to the office area K, to industrial area I, and also to the factory area P, as well as other places and locations. Rainwater enters the area's quaternary drainage system;
2. From the quaternary drainage system, the water then flows into the tertiary drainage channel (St), or some of it is collected first in a new polder (Po) to St;

3. Ideally, dirty water can be collected in the polder (Po) first, then cleaned in the waste water treatment (Tw), very emphasis is placed on areas that dispose of new liquid waste and then flow it to a clean water pond (Ka), and finally the new excess water in drainage to the sewer (can be via St, Ss, then Sp);
4. From the tertiary drainage system (St) is drained to the secondary drainage channel (Ss);
5. All flows from the secondary drainage channel are channeled to the primary drainage channel (Sp).

3. Research Method

3.1. Flow Chart

The method used to process the data in this paper is descriptive and quantitative methods, namely by collecting data in the field of primary and secondary data from relevant agencies, as well as collecting the literature related to this research. Data analyzed in such a manner consistent with the theory outlined above. Then made a final conclusion about the drainage pump capacity required to cope with floods in the region of East Surabaya with planning a return period of 2, 5, and 10 years.

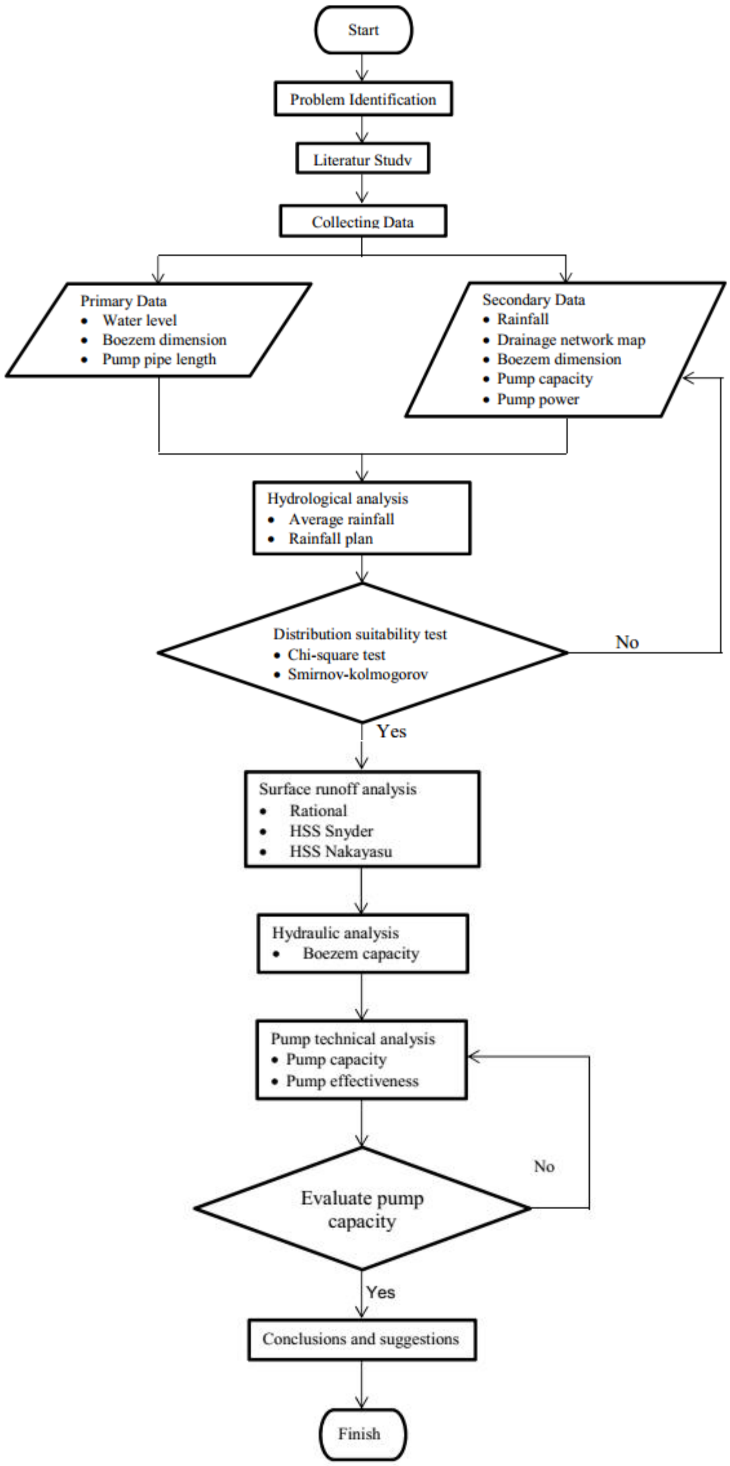

Figure 1. Research flow chart

3.2. Data Source

Table 1. Data source

No.	Data	Edition	Source	Typical
1	High water level	-	Survey	Primary data
2	Boezem dimention	-	Survey	Primary data
3	Pump pipe length	-	Survey	Primary data
4	Rainfall	2008-2018 (10 Years)	BMKG	Secondary data
5	Drainage network map	2013	DPUBMP	Secondary data
6	Boezem dimension	2016	DPUBMP	Secondary data
7	Pump capacity	2018	DPUBMP	Secondary data
8	Pump power	2018	DPUBMP	Secondary data

Source: Observations

4. Results and Discussion

There are 3 rain stations in the Kalidami watershed, namely: the Keputih rain station, the Gubeng rain station, and the Wonokromo rain station. In the calculation of the rainfall, not all areas of the rain station is used, only the territory contained in Kalidami DAS and affects the discharge occurred.

Table 2. Area

No.	Rain Station	Area (km^2)	Koefisien Thiessen (W)
1	Keputih	5.658	0.107
2	Gubeng	4.418	0.391
3	Wonokromo	1.209	0.501
	Total	11.285	

Source: (Sari et al., 2017)

The formula used is the method of calculating the average algebra / arithmetic, as well as the calculation method Thiessen polygons, whereas Isohyets method is not used because the nearest rain station used contained only 3 stations of rain, while the method requires at least 4 stations Isohyets rain.

Table 3. Maximum rainfall data

No.	Year	Keputih	Gubeng	Wonokromo	Arithmetic	Thiessen
1	2009	120	86	104	103.33	104.98
2	2010	90	106	110	102	98.41
3	2011	72	81	98	83.67	78.31
4	2012	85	70	106	87	81.38
5	2013	80	99	87	88.67	88.19
6	2014	134	109	83	108.67	118.75
7	2015	84	61	63	69.33	72.75
8	2016	164	98	108	123.33	132.16
9	2017	124	116	114	118	119.80
10	2018	49	65	73	62.33	57.83
					$\Sigma = 946.33$	$\Sigma = 952.55$
					$\bar{X} = 94.63$	$\bar{X} = 95.25$

Source: Processed data by researchers (2021)

The statistical parameters were calculated for the Normal method and the Gumbel method consisting of the skewness coefficient or CS, and the kurtosis coefficient or CK.

$$S_x = \sqrt{\frac{5057.67}{(10-1)}} = 23.71$$
$$C_s = \frac{10\ x\ 7288.88}{(10-1)(10-2)\ 23.71^3} = 0.076$$
$$C_k = \frac{10^2\ x\ 4871726.91}{(10-1)(10-2)(10-3)\ 23.71^4} = 3.06$$

Then calculated the skewness coefficient or CS and the kurtosis coefficient or CK for distribution log Pearson III.

$$S_x = \sqrt{\frac{0.1134}{(10-1)}} = 0.1122$$
$$C_s = \frac{10\ x\ (-0.0036)}{(10-1)(10-2)\ 0.1122^3} = -0.3535$$
$$C_k = \frac{10^2\ x\ 0.0028}{(10-1)(10-2)(10-3)\ 0.1122^4} = 3.4641$$

The following are the results of the inclination coefficient and the peak coefficient from the Normal method, the Gumbel method, and the Log Pearson method, as well as the required requirements:

Table 4. Distribution requirements

No.	Type	Requirements	Calculation	Result
1	Gumbel	cs = 1.139	cs = 0.076	Does not meet
		ck < 5.402	ck = 3.061	Meet requirements
2	Normal	cs = 0	cs = 0.076	Does not meet
		ck = 3	ck = 3.061	Meet requirements
3	Log pearson III	cs = exempt	cs = -0.354	Meet requirements
		ck = exempt	ck = 3.464	Meet requirements

Source: Processed data by researchers (2021)

According to the conclusion above, the design rainfall data that will be used is the Log Pearson III method. Then, the above data will be tested with two different kinds of distribution fit test.

Table 5. Recapitulation of distribution suitability test

Distribution equation	Suitability Test							
	Chi Kuadrat Test				Smirnov-Kolmogorov Test			
	X²	Nilai	Xh²	Evaluation	Dmax	Nilai	D0	Evaluation
Log Pearson III	1	<	5.99	Accepted	0.0565	<	0.41	Accepted

Source: Processed data by researchers (2021)

It can be concluded that the Log Pearson III method can be used as a design rainfall calculation. Then the analysis of surface runoff that occurs can be calculated using three methods. According to Surabaya Drainage Masterplan concentrated rainfall that occurred in Surabaya on average for 5 hours. With this centralized average rainfall, the value of the hourly rain pattern can be seen with the following results:

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Table 6. Hourly rain pattern

No.	Time	Hourly rain pattern				Return Period		
	T	Rt	t.Rt	(t-1).R(t-1)	R't	2	5	10
1	1	0.58	0.58	0	0.58	25.73	31.61	34.90
2	2	0.37	0.74	0.58	0.15	6.69	8.22	9.07
3	3	0.28	0.84	0.74	0.11	4.69	5.76	6.36
4	4	0.23	0.93	0.84	0.08	3.74	4.59	5.07
5	5	0.2	1	0.93	0.07	3.15	3.87	4.28

Source: Processed data by researchers (2021)

Based on the three methods of surface runoff analysis above, it can be seen the highest flood discharge of each method as follows:

Table 7. Recapitulation of maximum flood discharge

No.	Method	Maximum flood discharge		
		2	5	10
1	Rasional	30.0192	36.8710954	40.7120272
2	HSS Snyder	22.8682	28.08788272	31.0138506
3	HSS Nakayasu	29.7161	36.49888531	40.3010433

Source: Processed data by researchers (2021)

The highest flood discharge results are using the rational method, but because the rational method does not have a time calculation such as HSS Snyder and HSS Nakayasu, the HSS Nakayasu method will be used. Which can be seen in the chart below flood discharges occurring ratio based on the period over.

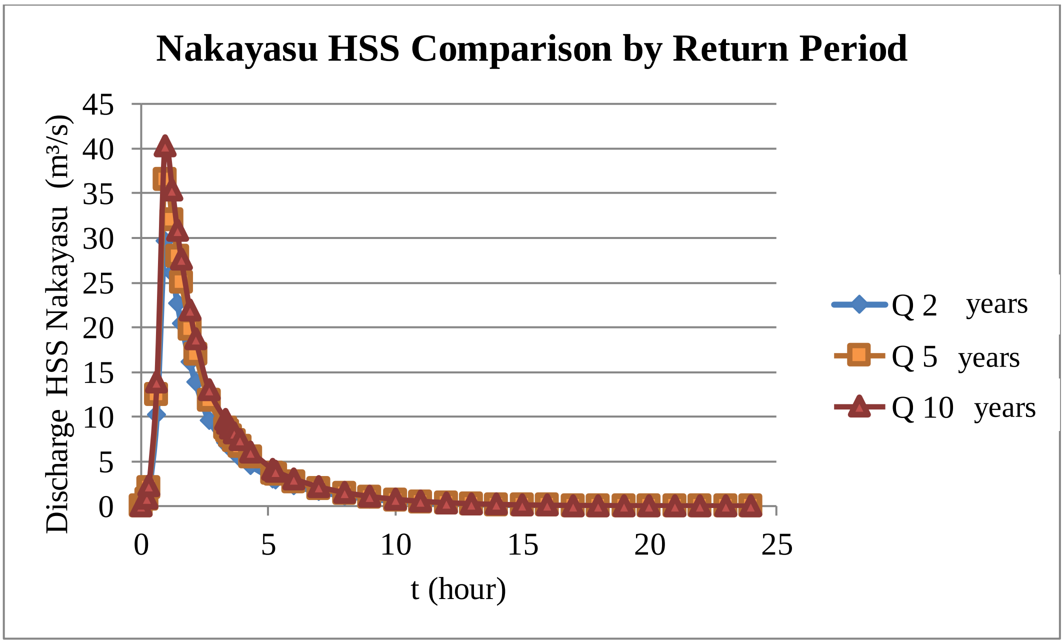

Figure 2. The comparison of the HSS Nakayasu flood discharge based on the return period

So if the pump house Boezem Kalidami has 7 pumps with each pump capacity is 3 m³/s, the total capacity of efficient amounted to 15.33 m³/s.

If recalculate flood routing by entering the pump efficiency as a factor in the calculation, the results will be as follows:

Table 8. Flood routing with a 3 m³/s pump with a return period of 10 years

No.	t	Δt	Q	Vin	n	Qpump	Vout	Vrunover	Annotation	H
1	0	0	0	0	0	0	0	0	-	-
2	0.23	835.37	0.91	761.32	0	0	0	-48679	Not runover	-
3	0.31	278.46	2.29	637.74	0	0	0	-48041	Not runover	-
4	0.62	1113.83	13.84	15412.43	0	0	0	-32629	Not runover	-
5	0.93	1113.83	40.30	44888.63	6	13.15	14641.2	-2381	Not runover	-
6	1.42	1779.04	30.84	54869.79	7	15.34	27282.7	25206	Run over	1.02
7	1.58	564.09	27.67	15610.48	7	15.34	8650.62	32165.86	Run over	1.30
8	2.14	2023.61	18.76	37959.04	7	15.34	31033.3	39091.55	Run over	1.58
9	2.67	1898.58	13.02	24726.89	7	15.34	29116	34702.49	Run over	1.40
10	3.32	2346.27	9.64	22624.46	7	15.34	35981.6	21345.37	Run over	0.86
11	3.41	330.46	9.24	3054.46	7	15.34	5067.83	19332	Run over	0.78
12	3.53	438.81	8.74	3834.20	7	15.34	6729.41	16437	Run over	0.66
13	4.32	2823.46	6.09	17182.70	6	13.14	37113.9	-3494.40	Not runover	-
14	5.18	3110.90	4.09	12709.08	5	10.95	34076.9	-24862	Not runover	-
15	5.28	348.07	3.91	1359.98	5	10.95	3812.8	-27315	Not runover	-
16	6	2595.23	3.04	7902.12	2	4.38	11371.3	-30784	Not runover	-
17	7	3600	2.15	7756.21	0	0	0	-23028	Not runover	-
18	8	3600	1.52	5488.18	0	0	0	-17540	Not runover	-
19	9	3600	1.08	3883.36	0	0	0	-13657	Not runover	-
20	10	3600	0.76	2747.81	0	0	0	-10909	Not runover	-
21	11	3600	0.54	1944.31	0	0	0	-8964.38	Not runover	-
22	12	3600	0.38	1375.76	0	0	0	-7588.61	Not runover	-
23	13	3600	0.27	973.47	0	0	0	-6615.14	Not runover	-
24	14	3600	0.19	688.81	0	0	0	-5926.33	Not runover	-
25	15	3600	0.14	487.39	0	0	0	-5438.93	Not runover	-
26	16	3600	0.10	344.87	0	0	0	-5094.06	Not runover	-
27	17	3600	0.07	244.03	0	0	0	-4850.03	Not runover	-
28	18	3600	0.05	172.67	0	0	0	-4677.36	Not runover	-
29	19	3600	0.03	122.18	0	0	0	-4555.18	Not runover	-
30	20	3600	0.02	86.45	0	0	0	-4468.73	Not runover	-
31	21	3600	0.02	61.17	0	0	0	-4407.56	Not runover	-
32	22	3600	0.01	43.28	0	0	0	-4364.27	Not runover	-
33	23	3600	0.01	30.63	0	0	0	-4333.64	Not runover	-
34	24	3600	0.01	21.67	0	0	0	-4311.97	Not runover	-

Source: Processed data by researchers (2021)

If it is planned to add up to 10 pumps with additional capacity of each pump up to 5 m³/s, but with a similar efficiency of 73.03%, the total efficient capacity of the planned pumps is 36.51 m³/s. By calculating the previous flood routing but replaced with a pump capacity of the pump capacity of the plan will result in the calculation as follows:

Table 9. Flood routing with a 5 m³/s pump with a return period of 10 years

No.	t	Δt	Q	Vin	n	Qpump	Vout	Vrunover	Annotation	H
1	0	0	0	0	0	0	0	0	-	-
2	0.23	835	0.91	761.32	0	0	0	-3550.65	Not run over	-
3	0.31	278	2.29	637.74	0	0	0	-2912.92	Not run over	-
4	0.62	1114	13.84	15412.43	4	14.61	16267.96	-3768.45	Not run over	-
5	0.93	1114	40.30	44888.63	10	36.51	40669.91	450.27	Run over	0.02
6	1.42	1779	30.84	54869.79	9	32.86	58462.99	-3142.94	Not run over	-
7	1.58	564	27.67	15610.48	7	25.56	14417.70	-1950.16	Not run over	-
8	2.14	2024	18.76	37959.04	6	21.91	44333.36	-8324.48	Not run over	-
9	2.67	1899	13.02	24726.89	5	18.26	34661.85	-18259.44	Not run over	-
10	3.32	2346	9.64	22624.46	4	14.61	34268.16	-29903.14	Not run over	-
11	3.41	330	9.24	3054.46	4	14.61	4826.50	-31675.19	Not run over	-
12	3.53	439	8.74	3834.20	4	14.61	6408.96	-34249.95	Not run over	-
13	4.32	2823	6.09	17182.70	4	14.61	41237.66	-58304.91	Not run over	-
14	5.18	3111	4.09	12709.08	0	0	0	-45595.83	Not run over	-
15	5.28	348	3.91	1359.98	0	0	0	-44235.85	Not run over	-
16	6	2595	3.04	7902.12	0	0	0	-36333.73	Not run over	-
17	7	3600	2.15	7756.21	0	0	0	-28577.52	Not run over	-
18	8	3600	1.52	5488.18	0	0	0	-23089.33	Not run over	-
19	9	3600	1.08	3883.36	0	0	0	-19205.98	Not run over	-
20	10	3600	0.76	2747.81	0	0	0	-16458.17	Not run over	-
21	11	3600	0.54	1944.31	0	0	0	-14513.86	Not run over	-
22	12	3600	0.38	1375.76	0	0	0	-13138.09	Not run over	-
23	13	3600	0.27	973.47	0	0	0	-12164.62	Not run over	-
24	14	3600	0.19	688.81	0	0	0	-11475.81	Not run over	-
25	15	3600	0.14	487.39	0	0	0	-10988.41	Not run over	-
26	16	3600	0.10	344.87	0	0	0	-10643.54	Not run over	-
27	17	3600	0.07	244.03	0	0	0	-10399.51	Not run over	-
28	18	3600	0.05	172.67	0	0	0	-10226.84	Not run over	-
29	19	3600	0.03	122.18	0	0	0	-10104.66	Not run over	-
30	20	3600	0.02	86.45	0	0	0	-10018.21	Not run over	-
31	21	3600	0.02	61.17	0	0	0	-9957.04	Not run over	-
32	22	3600	0.01	43.28	0	0	0	-9913.75	Not run over	-
33	23	3600	0.01	30.63	0	0	0	-9883.13	Not run over	-
34	24	3600	0.01	21.67	0	0	0	-9861.45	Not run over	-

Source: Processed data by researchers (2021)

With the plan to increase pump capacity and the number of pumps, there is a very significant difference in overflow, there is no overflow at all during the return period of 2 years and 5 years, and there is very little overflow, which is only 2 cm high during the 10 year return period.

5. Conclusion And Suggestions

5.1. Conclusion

1. Based on Nakayasu HSS method, the maximum discharge runoff that occurs during the period of 2, 5, and 10 years respectively is 29.72 m³ / s, 36.5 m³ / s, and 40.3 m³ / s.

2. 2 The volume of Boezem Kalidami reservoir is 49,440 m³ and with the planned runoff discharge, Boezem Kalidami can only accommodate runoff discharge up to the first 1.42 hours and there will be a runoff of 1.58 meters from Boezem Kalidami's lips at 2.14 hours.
3. 3 With a pipe length of up to 75 meters, the head loss that occurs in the pipe is large enough so that the efficiency obtained by the pump is only 73.03% with a total of 7 pumps so the total efficient capacity is only 15.34 m³ / s.

5.2. Suggestions

Planning for additional pump capacity and increasing the number of pumps in the Boezem Kalidami pump house is required. For the 10-year planning period, the pump capacity of 5 m³ / sec with the addition of up to 10 pieces of the pump is very inadequate.

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