ANALYSIS CAPACITY OF WATERWAY IN ANEKA ELOK EAST JAKARTA

Mohammad Imam Imamuddin[1], Laraswati Laraswati[2]
[1]Lecturer of Civil Engineering Study Program, University of Muhammadiyah Jakarta, INDONESIA
[2]Student of Civil Engineering Study Program, University of Muhammadiyah Jakarta, INDONESIA
E-mail: imamuddin002@gmail.com , laraswati96@gmail.com

ABSTRACT

The problem of flooding in Aneka Elok, which is located in the city of East Jakarta, is still a matter of concern, because every rainy season it is always routinely flooded. The catchment area in this study is 0,75 km2 has a primary waterway that flow along the Aneka Elok area which flows northward to the reservoir and pump house. The reservoir in Aneka Elok area is located on an area of 32.000m2, is able to accommodate a flood debit of 225m3 with an average depth of 4m. And has a pump capacity of 1000 liters/second as much as 2 units. With the current capacity of the reservoir and pump capacity, it is still not able to overcome the debit flood existing. This study aims to determine the existing debit in Aneka Elok area according to the return period of 5 years and 25 years, the debit flood existing in the research area, so that it can analyze the required waterway cross section and ideal pump capacity. With a return period of 5 years, the rainfall insensity of I5 is 91,95 mm/hour to calculate the debit flood due the rain and compared to the debit existing flood, it is found that thecapacity of the existing waterway at C1-C2, C4-C9 and C11 is not sufficient to accommodate the current debit flood, so it is necessary to modify the waterway dimension. With a return period of 25 years, the rainfall insensity I25 is 169,66 mm/hour to calculate the pump capacity, it is found that current capacity is still not sufficient so that it is necessary to increase the pump capacity from 2 m3/s to 3,5 m3/s to accelerate water flow.

Key word: flood; pudlles; drainage system; pump houses; pump capacity.

Received:	Revised:	Accepted:	Available online:
2021-08-27	2021-09-02	2021-10-05	2021-10-19

INTRODUCTION

Aneka Elok area is one of the area that is regularly flooded. There is one pump house in this area. Aneka Elok Pump House East Jakarta is located in Mahoni Elok Street, RT. 1/RW. 9. This area includes densely populated residential areas and housing area.

This Aneka Elok area has one of primary waterway along Mahoni Elok Street. It flows from the south side of the pump house to the north side of the floodgate. It is in the form of an open trapezoid with a retaining wall using river stones. This waterway towards the primary waterway which then leads to the waterway towards the reservoir or towards the pump house through the floodgate on the south side of the pump house (Asdak.C, 1995; Satriadi.I, 2017; Imamuddin.I, Chayanto.D, 2020; Yarsono.S, et.al, 2020).

Reservoir in Aneka Elok area has an area of 32.000 m2. With a catchment area of 0,75 km2. There is an article about Aneka Elok reservoir published on the wartakota website in 2016, that the average depth of the reservoir is 4 m with a capacity of up to 225 m3. Reservoir have an important role, that to accommodate the flow rate around the reservoir area for a while, then it will be flowed to the pump house.

Aneka Elok pump house has 2 units with a capacity of each flood pump is 1000 liters/second. This pump is to flow water from the retention pond in the pump house which is not accommodated in the reservoir to the outside of the Aneka Elok area.

For this problem, the writer wants to calculate the capacity of the waterway, reservoir, and ideal pump needs so that there is no puddle in Aneka Elok area.

pISSN 2302-4240
eISSN 2655-2086

Volume 10, No. 2, December 2021, pp.322-332
DOI: http://dx.doi.org/10.32832/astonjadro.v10i2

Research Purposes

The purpose of this study are as follows:

1. To determine the rainfall and planned debit flood for 5 years and 25 years return periods in Aneka Elok area.
2. To determine the debit flood existing in the research area.
3. To determine the ideal waterway dimensions.
4. To determine the ideal pump for overcoming run off

RESEARCH METHOD

Place and Time of Research

The research was conducted in Aneka Elok area with catchment area 0,75 km2. Which is carried out in October-November 2020.

Preparation Stages

Preparation stages is arranged for things that must be done with aim of time and work effectiveness.

1. Determine data requirements.
2. Literature study on the theoretical basis related to the research topic.
3. List related agencies that need correspondence which is used as a data source.
4. Site survey to get an overview of field conditions.

Primary Data

Primary data is a data that obtained directly in the field, that is:

1. Data topographic.
2. Data collecting on drainage waterway to reservoir.
3. Capacity data for pump and pump house.
4. Documentation.

Secondary Data

Secondary data obtained from agencies that has relevance in terms of planning, controlling and handling flood, including as follows:

1. Determine catchment area.
2. Rainfall data.
3. Determine the land use map.

Analysis and Data Processing

Stages of analysis and data processing used in this study are as follows:

1. Hydrology analysis
   It is used to determine the planned debit flood. The data used is rainfall data.
2. Hydrolic analysis
   It is used to find a waterway cross-section that can accommodate the planned debit flood.

Flowchart Research

In order to make it easy the research, it is necessary to make a flowchart as follows:

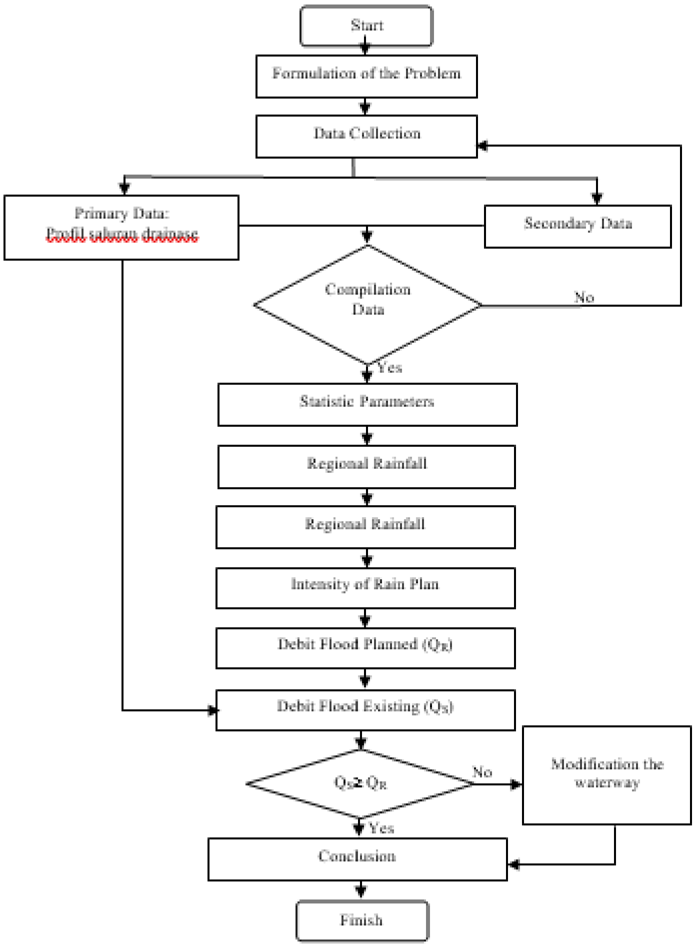

Figure 1. Flowchart

RESULT AND DISCUSSION

Rainfall Analysis

For the highest rainfall in 2020 obtained from online news published by kumparan.com/kumparansains with the title “BMKG: Extreme Rainfall in Jakarta Affected by Climate Change” which was published on January 3, 2020, and got the highest rainfall in 2020 at UPT Halim Perdana Kusuma Station was 377 mm/day

Table 1. Data maximum daily rainfall

Year	Data Maximum Daily Rainfall (mm)											
	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DES
2009	7.8	26	16	140.4	64.2	19.2	0	95	0	20.1	82.1	24
2010	86.4	61.3	31.7	40	83.1	34.7	45	37.8	95	96.8	50.1	37.1
2011	21.6	305	23.5	19.4	64.2	18	12.3	0	5.6	32.4	89.6	22
2012	85.3	90.6	59.8	42	36.3	49.3	0.8	0	0	39.4	52.5	94.4
2013	161	43.8	87.7	73.7	66.3	18.6	67	0.2	3.2	24.5	66	31.8
2014	120.8	102.7	100.8	37.4	47.7	54.5	60.9	42.7	18.5	0	57.8	69.5
2015	67	124.6	124.5	92.6	26.8	28	0	1.7	0	1.2	36	80.6
2016	61.2	62.4	0	111.6	54	85.6	41.8	51.4	78.4	54.2	79.6	37.8

pISSN 2302-4240
eISSN 2655-2086
Volume 10, No. 2, December 2021, pp.322-332
DOI: http://dx.doi.org/10.32832/astonjadro.v10i2

2017	43	136.3	42.2	91.6	26.2	24.4	4.9	19	2.6	55	39	62
2018	90.6	47.9	42	101.2	41.2	16.1	1,4	0	0	62.7	71.6	34.7
2019	-	-	-	196,00	-	-	-	0,00	1,00	-	-	229,00

Source: BMKG UPT Halim Perdana Kusuma Station

Table 2. Distribution calculation

Year	X_i	X_i-$\bar{X}$	$(X_i-\bar{X})^2$	$(X_i-\bar{X})^3$	$(X_i-\bar{X})^4$
2009	140,4	-26,11	681,65	-17796,62	464640,00
2010	96,4	-69,71	4859,25	-338730,34	23612327,43
2011	305	138,49	19179,94	2656262,10	367870165,00
2012	94,4	-72,11	5199,61	-374935,34	27035962,21
2013	161	-5,51	30,34	-167,13	920,62
2014	120,8	-45,71	2089,25	-95496,21	4364972,82
2015	124,6	-41,91	1756,31	-73603,96	3084619,21
2016	111.6	-54,91	3014,93	-165544,51	9089773,17
2017	136,3	-30,21	912,54	-27566,42	832735,46
2018	101,2	-65,31	4265,18	-278551,69	18191746,81
2019	229,00	62,49	3905,21	244042,98	15250652,67
2020	377,00	210,49	44306,74	9326199,91	1963087363,27
Total	1.998,10	0,00	90.200,95	10.854.112,78	2.432.885.878,67
$\bar{X}$	166,51				

From the calculation results above then determine the type of distribution, by looking for the standard deviation value (Sd), coefficient of variation (Cv), coefficient of skewness (Cs), dan coefficient of kurtosis (Ck).

$\bar{X}$ = 166,51 mm

S = 90,55

Cv = 0,54

Cs = 1,59

Ck = 5,26

Selection Type Distribution

Table 3. Summary of value Cv, Cs and CkDistribution Method

Distribution	Requirement	Result	Conclusion
Normal	Cs = 0	Cs = 1,59	Not eligible
	Ck = 3	Ck = 5,26	Not eligible
Gumbel	Cs ≤ 1,1396	Cs = 1,59	Not eligible
	Ck ≤ 5,4002	Ck = 5,26	Eligible
Log Normal	Cs = 3	Cs = 1,59	Not eligible
	Cs = 3 Cv	Cv = 0,54	Not eligible
Log Pearson III	Flexible	Cs = 1,59	Eligible
	Flexible	Ck = 5,26	Eligible

Source: SNI 2415:2016

From the table above, it can be seen that Cs and Ck value that meet the requirements is found in Log Pearson III distribution so that rainfall plan value is used for calculation the using Log Pearson III method.

Calculation of Average Rainfall Plan

Table 4. Calculation of Log Pearson III

Year	X_i	Log Xi	(Log X_i – Log $\bar{X}$)^2	(Log X_i - Log $\bar{X}$)^3	(Log X_i- Log $\bar{X}$)^4
2009	2,147	0,147	0,056481	0,02	2009
2010	1,986	0,049	0,010969	0,00	2010
2011	2,484	0,519	0,374199	0,27	2011
2012	1,975	0,045	0,009432	0,00	2012
2013	2,207	0,196	0,087019	0,04	2013
2014	2,082	0,101	0,032272	0,01	2014
2015	2,096	0,1101	0,036538	0,01	2015
2016	2,048	0,081	0,022900	0,01	2016
2017	2,134	0,1375	0,050985	0,02	2017
2018	2,005	0,058	0,014083	0,00	2018
2019	2,360	0,355	0,211865	0,13	2019
2020	2,576	0,660	0,536679	0,44	2020
Total		26,100	0,43011	0,07741	0,04035
Log $\bar{X}$		2,175			

From this calculation, the calculate standard deviation (Sd), coefficient of skewness (Cs), and coefficient of kurtosis (Ck).

$\bar{X}$ = 2,175 mm

S = 0,22

Cs = 0,81

Ck = 2,57

With Cs = 0,81 then the interpolation will be calculated from 0,8-0,9.

Table 5. Factor frequencyfor distribution Log Pearson III

Year	Log $\bar{X}$	K		
		0,80	0,81	0,90
5	2,175	0,780	0,779	0,769
25	2,175	1,993	1,996	2,018

Table 6. Calculation distribution of rain with Log Pearson III

Year	Log $\bar{X}$	K	Log X_T	K_T
5	2,175	0,779	2,345	221,455
25	2,175	1,996	2,611	408,631

Calculation of Time Concentration (Tc)

The slope waterway is calculated by the following equation:

ASTONJADRO: Jurnal Rekayasa Sipil
pISSN 2302-4240
eISSN 2655-2086
Volume 10, No. 2, December 2021, pp.322-332
DOI: http://dx.doi.org/10.32832/astonjadro.v10i2

$$S0 = \left(\frac{7{,}92-5{,}18}{1160}\right) = 0{,}002$$

Time concentration can be calculated by the following equation:

$$Tc = 0{,}0195\left(\frac{1160}{\sqrt{0{,}002}}\right)^{0{,}77} = 45{,}79 \text{ minutes} = 0{,}763 \text{ hours}$$

Calculation of Rain Intensity

a. Rain intensity period 5 years

$$I_5 = \frac{221{,}45}{24}\left(\frac{24}{0{,}763}\right)^{\frac{2}{3}} = 91{,}95 \text{ mm/hour}$$

b. Rain intensity period 25 years

$$I_{25} = \frac{408{,}63}{24}\left(\frac{24}{0{,}763}\right)^{\frac{2}{3}} = 169{,}66 \text{ mm/hour}$$

Calculation Average Coefficient of Flow

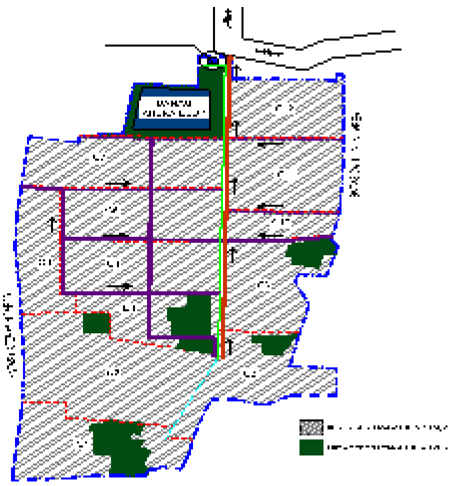

Figure 2. Distribution of catchment area Source: Google Earth Pro

Table 7. Average coefficient run off

Catchment Area		A (km^2)	C	A x C	Catchment Area		A (km^2)	C	A x C
C1	$C_{average}$			0,806	C7	$C_{average}$			0,891
	Residential house	0,0475	0,90	0,0427		Residential house	0,0570	0,90	0,0513
	Garden	0,0101	0,22	0,0022		Garden	0,0023	0,22	0,0005
	Pavement	0,0102	0,95	0,0097		Pavement	0,0167	0,95	0,0159
C2	$C_{average}$			0,905	C8	$C_{average}$			0,783
	Residential house	0,0821	0,90	0,0739		Residential house	0,0413	0,90	0,0372
	Garden	0,0011	0,22	0,0002		Garden	0,0107	0,22	0,0023
	Pavement	0,0253	0,95	0,0240		Pavement	0,0070	0,95	0,0067
C3	$C_{average}$			0,741	C9	$C_{average}$			0,832
	Residential house	0,0254	0,90	0,0229		Residential house	0,0432	0,90	0,0388
	Garden	0,0120	0,22	0,0026		Garden	0,0082	0,22	0,0018

	Pavement	0,0105	0,95	0,0100		Pavement	0,0172	0,95	0,0163
C4	$C_{average}$			0,883	C10	$C_{average}$			0,891
	Residential house	0,0389	0,90	0,0350		Residential house	0,0163	0,90	0,0146
	Garden	0,0021	0,22	0,0005		Garden	0,0007	0,22	0,00014
	Pavement	0,0109	0,95	0,0103		Pavement	0,0048	0,95	0,0045
C5	$C_{average}$			0,888	C11	$C_{average}$			0,891
	Residential house	0,0436	0,90	0,0392		Residential house	0,0478	0,90	0,0430
	Garden	0,0020	0,22	0,0004		Garden	0,0019	0,22	0,0004
	Pavement	0,0125	0,95	0,0119		Pavement	0,0140	0,95	0,0133
C6	$C_{average}$			0,885	C12	$C_{average}$			0,891
	Residential house	0,0359	0,90	0,0323		Residential house	0,0381	0,90	0,0343
	Garden	0,0018	0,22	0,0004		Garden	0,0015	0,22	0,0003
	Pavement	0,0102	0,95	0,0096		Pavement	0,0112	0,95	0,0106
C13	$C_{average}$			0,749					
	Residential house	0,0028	0,90	0,0025					
	Garden	0,0088	0,22	0,0019					
	Pavement	0,0209	0,95	0,0199					

Calculation of Debit Flood Due to Rain (Qt)

For calculation of debit rain plan with catchment area that is affected by another debit catchment area, then the result is added with that debit catchment area.

Debit plan uses rainfall intensity for 5 years return period obtained by the following calculations:

$Q_{C_1} = \frac{1}{3,6}$ = x 0,806 x 91,946 mm/hour x 0,0678 km2 = 1,396 m3/s

$Q_{C_2} = \frac{1}{3,6}$ = x 1,905 x 91,946 mm/hour x 0,1085 km2 + Q_{C_1} = 3,903 m3/s

$Q_{C_3} = \frac{1}{3,6}$ = x 0,741 x 91,946 mm/hour x 0,048 km2 + 0,75 Q_{C_2} = 3,835 m3/s

$Q_{C_4} = \frac{1}{3,6}$ = x 0,883 x 91,946 mm/hour x 0,0518 km2 + Q_{C_3} = 5,004 m3/s

$Q_{C_5} = \frac{1}{3,6}$ = x 0,888 x 91,946 mm/hour x 0,0581 km2 +Q_{C_4} = 5,22 m3/s

$Q_{C_6} = \frac{1}{3,6}$ = x 0,885 x 91,946 mm/hour x 0,0479 km2 + 0,25Q_{C_2} = 2,058 m3/s

$Q_{C_7} = \frac{1}{3,6}$ = x 0,891 x 91,946 mm/hour x 0,076 km2 $Q_{C_5} + Q_{C_6}$ = 9,007 m3/s

$Q_{C_8} = \frac{1}{3,6}$ = x 0,783 x 91,946 mm/hour x 0,059 km2 = 1,18 m3/s

$Q_{C_9} = \frac{1}{3,6}$ = x 0,832 x 91,946 mm/hour x 0,0685 km2 + Q_{C_8} = 2,635 m3/s

$Q_{C_{10}} = \frac{1}{3,6}$ = x 0,891 x 91,946 mm/hour x 0,0217 km2 + Q_{C_9} = 3,128 m3/s

$Q_{C_{11}} = \frac{1}{3,6}$ = x 0,891 x 91,946 mm/hour x 0,0637 km2 + $Q_{C_{10}}$ = 4,577 m3/s

$Q_{C_{12}} = \frac{1}{3,6}$ = x 0,891 x 91,946 mm/hour x 0,0508 km2 + $Q_{C_{11}}$ = 5,733 m3/s

$C_{C_{13}} = \frac{1}{3,6}$ = x 0,749 x 91,946 mm/hour x 0,0325 km2 + $Q_{C_{12}}$ = 6,353 m3/s

Volume 10, No. 2, December 2021, pp.322-332
DOI: http://dx.doi.org/10.32832/astonjadro.v10i2

$C_{14} = Q_{C_7} + Q_{C_{13}}$ = 15,361 m3/s

Calculation of Existing Debit Flood (Qs)

Table 8. Calculation of existing debit flood (Qs)

Direction Flow	Profie Name	CA	Waterway Section (m) b	h	z	m	Q_s (m^3/s)
↓	A	C1	1	0,5			0,709
	B	C2	1	1,5			2,009
	A	C3	1	0,5			1,115
	C	C4	0,6	0,7			0,582
	C	C5	0,6	0,7			0,748
	D	C6	0,45	0,4			0,253
	C	C7	0,6	0,7			0,625
	A	C8	1	0,5			0,951
	D	C9	0,45	0,4			0,109
	C	C10	0,6	0,7			0,731
	C	C11	0,6	0,7			1,046
	C	C12	0,6	0,7			3,126
	E	C13	4,2	2,1	7,2	3,303	49,528

The example of calculation existing debit flood:

a. Capacity of existing waterway C1

A = 1 m x 0,5 m = 0,5 m2

P = b + 2h = 1 m + (2 x 0,5 m) = 2 m

R = A/P = 0,5 m2 / 2 m = 0,25 m

V $= \frac{1}{0{,}014} x\ 0{,}25^{\frac{2}{3}} x\ 0{,}0025^{\frac{1}{2}}$ = 1,417 m/s

QC1 = 0,5 m2 x 1,417 m/s = 0,709 m3/s

b. Capacity of existing waterway C13

A = x (7,2 m + 4,2 m) x 2,1 m = 11,97 m2

y $= \sqrt{[(0{,}5\ x\ 7{,}2)^2 - (0{,}5\ x\ 4{,}2)^2] + 2{,}1^2}$ = 3,303

P = b + 2y = 1 m + (2 x 3,303 m) = 13,807 m

R = A/P = 11,97 m2 / 13,807 m = 0,867 m

V $= \frac{1}{0{,}013} x\ 0{,}867^{\frac{2}{3}} x\ 0{,}0035^{\frac{1}{2}}$ = 4,138 m/s

QC13= 11,97 m2 x 4,138 m/s = 49,528 m3/s

Then presented in table 8 above.

Comparison Existing Waterway Capacity with Debit Due to Rainfall

Table 9. Comparison Qt and Qs

Direction Flow	Profie Name	CA	Waterway Section (m) b	h	z	m	Q_t (m^3/s)	Q_s (m^3/s)	Description
↓	A	C1	1	0,5			1,396	0,709	Not save
	B	C2	1	1,5			2,508	2,009	Not save
	A	C3	1	0,5			0,909	1,115	Save
	C	C4	0,6	0,7			1,169	0,582	Not save
	C	C5	0,6	0,7			1,318	0,748	Not save
	D	C6	0,45	0,4			1,083	0,253	Not save
	C	C7	0,6	0,7			1,730	0,625	Not save
	A	C8	1	0,5			1,180	0,951	Not save
	D	C9	0,45	0,4			1,456	0,109	Not save
	C	C10	0,6	0,7			0,494	0,731	Save
	C	C11	0,6	0,7			1,450	1,046	Not save
	C	C12	0,6	0,7			1,156	3,126	Save
	E	C13	4,2	2,1	7,2	3,303	0,621	49,528	Save

Comparison Debit Due to Rainfall with Debit Waterway Plan

Table 10. Comparison Qt and Qsr

Direction Flow	Profile Name Old	New	CA	Waterway Section (m) Old b	h	z	m	New b	h	Qt (m^3/s)	Qs (m^3/s)	Desc
↓	A	F	C1	1	0,5	-	-	1,2	1	1,396	2,229	Save
	B	G	C2	1	1,5	-	-	1,2	1,5	2,508	2,635	Save
	A	-	C3	1	0,5	-	-	-	-	0,909	1,115	Save
	C	H	C4	0,6	0,7	-	-	0,8	1	1,169	1,361	Save
	C	I	C5	0,6	0,7	-	-	0,8	0,8	1,318	1,337	Save
	D	I	C6	0,45	0,4	-	-	0,8	0,8	1,083	1,358	Save
	C	J	C7	0,6	0,7	-	-	0,9	1	1,730	1,738	Save
	A	J	C8	1	0,5	-	-	0,9	1	1,180	1,977	Save
	D	K	C9	0,45	0,4	-	-	1	1,5	1,456	1,725	Save
	C	-	C10	0,6	0,7	-	-	-	-	0,494	0,731	Save
	C	L	C11	0,6	0,7	-	-	0,8	0,9	1,450	1,507	Save
	C	-	C12	0,6	0,7	-	-	-	-	1,156	3,126	Save
	E	-	C13	4,2	2,1	7,2	3,303	-	-	0,621	49,528	Save

Surface Water Volume

In order to determine the need for the pump used, it is necessary to know the volume of surface water that must be streamed by pump, using the rainfall intensity for 25 years return period.

Survace water volume = 0,2778 x C x I25 x A x T

Information:

C = Flow coefficient

I25 = Rainfall intensity for 25 years return period (m/s)

A = Area (m2)

T = Duration of average maximum rainfall (s)

Survace water volume = 0,2778 x 0,855 x 0,000047 x 754000 x 3600 = 30367,2 m3

Volume 10, No. 2, December 2021, pp.322-332
DOI: http://dx.doi.org/10.32832/astonjadro.v10i2

Volume Storage Capacity

Vpool 1 = 1 x 6,3 x 5 x 13 = 409,5 m3

Vpool 2 = 1 x 9,5 x 5x 8,92 = 423,7 m3

Vwaterway to reservoir = 0,5 x (7,5+5) x 5 x 67 = 2093,75 m3

Vreservoir = 225 m3

Table11. Volume of waterway storage capacity

Profile Name	Waterway Section (m) b	h	z	L	n	V (m^3)
A	1	0,5		379,26	1	189,63
C	0,6	0,7		1006,71	2	845,636
E	4,2	2,1	7,2	891,99	1	10677,1
F	1,2	1		189,72	1	227,664
G	1,2	1,5		288,63	1	519,534
H	0,8	1		289	1	231,2
I	0,8	0,8		707,97	2	906,202
J	0,9	1		634,91	2	1142,84
K	1	1,5		399,57	1	599,355
L	0,8	0,9		243	1	174,96
					Total	15514,1

Total volume of water pumped:

$V_{waterway\ capacity}$ = $V_{waterway} + V_{pool\ 1} + V_{pool\ 2} + V_{waterway\ to\ reservoir}$ + Vreservoir

= 15514,1 + 409,5 + 423,7 +2093,75 + 225 = 1866,1 m3

$V_{water\ pumped}$ = $V_{surface\ water} - V_{waterway\ capacity}$

= 30367,2 m3 – 1866,1 m3 = 11701,2 m3

Pump Capacity

The pump capacity requirement is obtained by the equation as follows:

$$Qp = \frac{\text{Volume of water pumped}}{T} = \frac{11701,2}{3600} = 3,25\ m3/s$$

Capacity required pump = 3,25 m3/s

Existing pump capacity = 2 m3/s

Additional pump

= cap. required pump – cap. existing pump

= 3,25 m3/s – 2 m3/s = 1,25 m3/s ≈ 1,5 m3/s

Additional pump capacity is required:

1 m3/s = 1000 liters/s (1 unit)

0,5 m3/s= 500 liters/s (1 unit)

CONCLUSION

Waterway capacity is not enough to accommodate rainfall 5 years return period, so modifications to the existing waterway are carried out on C1 - C2, C4 – C9 and C11 waterways. Pump capacity is not sufficient, additional pumps with a capacity of 1.5 m3/s are needed, with an existing capacity of 2 m3/s. so the total pump at Aneka Elok Pump House is 3,5 m3/s.

REFERENCES

Asdak, C. (1995). Hidrologi dan pengelolaan Daerah Aliran Sungai. Gajah Mada University Press. Yogyakarta. (Indonesian).

Badan Standarisasi Nasional. (2016). SNI 2415:2016 Tata Cara Perhitungan Debit Banjir Rencana. Jakarta. (Indonesian).

Hasmar, H.A. Halim. (2011). Drainase Terapan.UII Press. Yogyakarta. (Indonesian).

I Satriadi, 2017. ANALISIS HIDROGRAF BANJIR SALURAN IRIGASI CIBALOK BOGOR, ASTONJADRO: JURNAL REKAYASA SIPIL 6 (1), 49-59. (Indonesian). http://ejournal.uika-bogor.ac.id/index.php/ASTONJADRO/article/viewFile/2261/1435

Kementerian Pekerjaan Umum. (2013). Tata Cara Perencanaan, Pelaksanaan, Operasi dan Pemeliharaan Sistem Pompa. Direktorat Jenderal Cipta Karya. Jakarta. (Indonesian).

Linsley, Ray K. (1986). Hidrologi Untuk Insinyur. Erlangga. Jakarta. (Indonesian).

M Imamuddin, D Cahyanto, 2020. ANALYSIS OF DRAINAGE CHANNEL CAPACITY AT SINDANG STREET IN SINDANG HOUSE PUMP AREA. ASTONJADRO: JURNAL REKAYASA SIPIL 9 (2), 132-144. http://ejournal.uika-bogor.ac.id/index.php/ASTONJADRO/article/view/3387

Republik Indonesia. (2012). Peraturan Pemerintah Republik Indonesia No. 37 Tahun 2012 tentang Pengelolaan Daerah Aliran Sungai. Lembaran Negara RI Tahun 2012, No. 62. Sekretariat Negara. Jakarta. (Indonesian).

Soemarto, C. D. (1995). Hidrologi Teknik. Erlangga. Jakarta. (Indonesian).

Suripin. (2004). Sistem Drainase Perkotaan yang Berkelanjutan. Edisi Pertama. Andi Offset. Yogyakarta. (Indonesian).

Triatmodjo Bambang. (2008). Hidrolika Terapan. Beta Offset. Yogyakarta. (Indonesian).

Wesli. (2008). Drainase Perkotaan. Edisi Pertama. Graha Ilmu. Yogyakarta. (Indonesian).

Fatrhur Reza Al Fatoni, Soebagiono. (2019). “Kajian Pompa Banjir pada Kali Tebu Tambak Wedi Surabaya”. Jurnal Rekayasa dan Manajemen Konstruksi, Vol. 7, No.2, Hal. 93. (Indonesian).

Kamilia Aziz, S. (2011). “Pola Pengendalian Banjir pada Bagian Hilir Saluran Primer Wonorejo”. Jurnal Aplikasi ISSN, 1907-753X, Volume 9, Nomor 2, Halaman 33. (Indonesian).

S Yarsono, M Imamuddin, J Suwandi, LM TN, I Wulandari, B Al Hanif, Eva Nur Septinia, Giman Giman, 2020. PENGARUH PERUBAHAN DIMENSI TANGKI TERHADAP DESAIN PONDASI EKSISTING TANGKI REAKTOR PILOT PLANT BIOGAS POME. ASTONJADRO: JURNAL REKAYASA SIPIL 9 (1), 24-29. (Indonesian). http://ejournal.uika-bogor.ac.id/index.php/ASTONJADRO/article/view/2823