FLOOD MANAGEMENT IN THE AREA OF PEJATEN EDUCATIONAL AND CULTURAL COMPLEX SOUTH JAKARTA

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ABSTRACT

The Educational and Cultural Complex Area Waterway is located in Pasar Minggu District. There is connecting Waterway (PHB Pulo), which floods every rainy season. It is due to the non-technical cross-section of the waterway, sedimentation, and garbage on the west side. The obstructed flow direction is found in the elegant Pejaten housing, where the high water level of PHB Pulo causes the water to slow down. From the above conditions, it is necessary to normalize the cross-section of 1 x 1 m^2 using concrete / u-ditch to accommodate five-year rainfall of 2,305 m^3/second, with a pump capacity to accelerate the flow to PHB Pulo by 1 m3/second.

Keywords: Pulo PHB waterway, Flood, Pump Capacity.

INTRODUCTION

The Pejaten Educational and Cultural complex area (the complex area) of South Jakarta has a lower contour than the Pejaten Raya road. In the existing condition, there is a Rambutan road waterway on the east side of the complex, and the Al-Hidayah Mosque road waterway, on the west side. The Sawo and Jamlang road channels, located on the south side of the complex, flow into the complex area and then rush to the north side.

Masjid Al-Hidayah road is a high road, so the water flow from the east side leads to the complex, while the flow from the west side leads to the river / PHB Pulo.

The water flow from the complex area leads to the Elegant Pejaten Housing via Jalan Malinjo and empties into PHB Pulo.

In recent years the complex area has experienced flooding due to the high water level of PHB Pulo. A waterway enters the residents' houses in the Elegant Pejaten housing so that the flow from the complex area becomes blocked. It is exacerbated by sedimentation and garbage in the waterway, making the water recedes longer.

The purposes of the analysis are:

1. To know the debit design of the existing drainage waterway at the site.
2. To find out whether the capacity of the existing drainage waterway can accommodate the debit design of the 25 years return period or not.
3. To obtain a capacity that can accommodate flood discharge with a return period of 25 years.
4. To know the required pump capacity.

THEORETICAL BASIS

Watershed

A Watershed is a land area that is an integral part of the river and its tributaries that function to accommodate, store and drain water. A watershed on a small scale is called a catchment area, a land area bounded by a ridge or topographic dividing boundaries, which functions to receive, store, and drain the rainfall that falls on it[1].

Flood and Inundation

Flooding and inundation are conditions where the water cannot be accommodated in the sewer or blocked water flow. It overflows and inundates the surrounding area[2].

Drainage System

Drainage means to drain, drain, dispose of, or divert water. In general, drainage can be defined as a technical action to reduce excess water, either from rainwater, seepage, or excess irrigation water from an area/land, so that the function of the area/land is not disturbed[3]. Drainage can also be interpreted as an attempt to control groundwater quality concerning salinity. Thus, drainage concerns not only surface water but also groundwater[4].

Return Period Hydrological Design (T)

Table 1

Hydrological design criteria of urban drainage systems

City Typology	Catchment Area (Ha) < 10	10 – 100	101 – 500	> 500
Metropolis	2 Years	2 Years – 5 Years	5 Years – 10 Years	5 Years – 10 Years
Big city	2 Years	2 Years – 5 Years	2 Years – 5 Years	5 Years – 10 Years
Medium City	2 Years	2 Years – 5 Years	2 Years – 5 Years	5 Years – 10 Years
Small town	2 Years	2 Years	2 Years	2 Years

Source: [5]

Probability Distribution

a. Average Value

$\overline{X} = \sum_{i=1}^{n} Xi/n$ .... (1)

where:

$\overline{X}$ = Average Value

Xi = Variant value

n = Number of data

b. Standard Deviation

$S_x = \sqrt{\frac{\sum_{i=1}^{n}(Xi-\overline{X})^2}{n-1}}$ ………..…………… (2)

where:

S_x = Standard deviation

$\overline{X}$ = Average value

Xi = Variant Value

n = Number of data

c. Variation Coefficient

$Cv = \frac{S_x}{\overline{X}}$ ……………………………… (3)

where:

Cv = Variation coefficient

S_x = Standard deviation

$\overline{X}$ = Average value

d. Slope Coefficient

$Cs = \frac{n \text{ x } \sum_{i=1}^{n}(Xi-\overline{X})^3}{(n-1) \text{ x } (n-2) \text{ x } S_x^{\,3}}$ ………………... (4)

where:

Cs = Slope coefficient/skewness

S_x = Standard deviation

n = Number of data

Data Distribution Type

Table 2

Statistical parameter requirements of a distribution

No.	Distribution	Requirement
1	Gumbel	Cs = 1.14 Ck = 5.4
2	Normal	Cs = 0 Ck = 3
3	Log Normal	Cs = Cv3 + 3Cv Ck = Cv8 + 6Cv6 + 15Cv4 + 16Cv2 + 3
4	Log Pearson III	Apart from the above values

Gumbel Method

$X_t = \overline{X} + \frac{(Y_t - Y_n)}{S_n} . S_x$ …………………. (5)

where:

X_t = Rainfall plan (mm)

Y_t = *Reduced variate* of Gumbel parameter

Y_n = *Reduced mean*

a. Log Pearson III Distribution Method

$\text{Log } X_t = \text{Log } \overline{X} + K_t . S_x$ …………….. (6)

where:

$\text{Log } X_t$ = Rainfall plan (mm)

$\text{Log } \overline{X}$ = Average value of rainfall plan (mm)

K_t = Frequency factor (Log Pearson III)

S_x = Standard deviation

Plan Waterway Debit ($Q_{Hydrology}$)

a. ***Rational Method***

Q = 0,278.C.I.A ……………….....….… (7)

where:

Q = Flood debit (m^3/sec)

C = Run off coefficient

I = Rainfall intensity (mm/hour)

A = The size of the catchment area (km^2)

b. ***Runoff Coefficient***

$C_{\text{rata-rata}} = \frac{\sum_{i=1}^{n} C_i A_i}{\sum_{i=1}^{n} A_i}$ ………………..... (8)

where:

C = Runoff coefficient value

A = The size of the catchment area (km^2)

c. ***Concentration Time***

$T_c = \frac{0{,}0195 . L^{0{,}77}}{S^{0{,}385}}$ ………………….…. (9)

Dimana:

Tc = Concentration time (minute)

L = Water track length (m)

S = Waterway slope

d. ***Concentration Time***

$I = \frac{R_{24}}{24} \left[\frac{24}{T_c}\right]^{2/3}$ ………………………. (10)

R_{24} = Rainfall in 24 hours (mm)

T_c = Concentration Time (hour)

I = Rainfall intensity (mm/hour)

Existing Waterway Debit ($Q_{Hydraulic}$)

a. ***Waterway Debit***

Q = A . V ……………..……….… (11)

where:

Q = Waterway debit (m^3/second)

A = Wet cross-sectional area (m^2)

V = Average flow speed (m/second)

b. ***Flow Speed***

$V = \frac{1}{n} . R^{2/3} . S^{1/2}$ …………………..….. (12)

where:

V = Average flow speed (m/second)
n = Manning roughness coefficient
R = Hydraulic spokes (m)
S = waterway slope

c. ***Rainfall Intensity***

$I = \frac{R_{24}}{24}\left[\frac{24}{T_c}\right]^{2/3}$ ……………………………… (13)

where:

R_{24} = Plan rainfall in 24 hours (mm)
T_c = Concentration time (hour)
I = Rainfall intensity (mm/hour)

RESEARCH METHOD

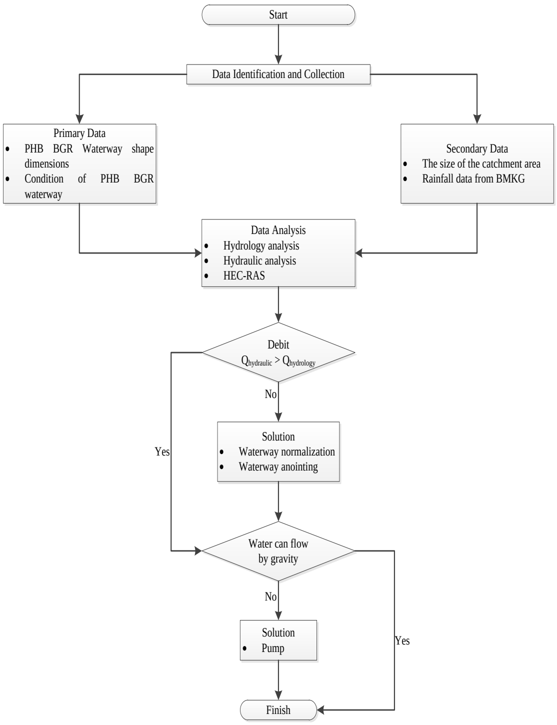

Figure 2. Stages of research methodology

RESULTS AND DISCUSSION

Return Period Hydrological Design (T)

The complex area based on Table 1 with an area of 0.31 km^2 uses a return period (T) of five years[6].

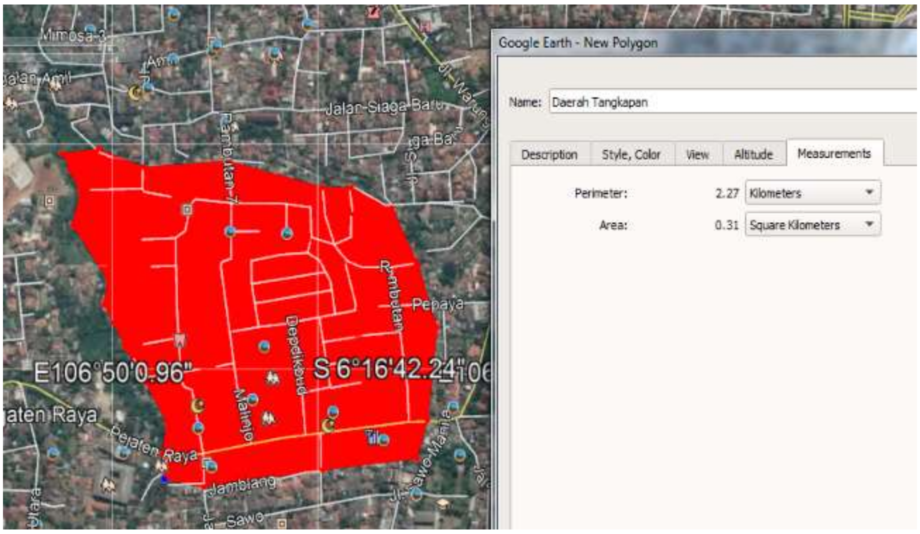

Figure 3. Size of catchment area

Probability Distribution

a. Maximum rainfall using Halim Perdana Kusuma's precipitation

Table 3

Maximum rainfall

No	Year	Rainfall
1	2006	88.5
2	2007	217.6
3	2008	136.1
4	2009	140.4
5	2010	95
6	2011	89.6
7	2012	59.8
8	2013	66
9	2014	120.8
10	2015	72
11	2016	111.6
12	2017	91.6
13	2018	101.2
14	2019	88.5
15	2020	377
	Total	1855.7
	Average	123.713333

Source: BMKG Halim

Table 4

Calculation of rainfall distribution

No	Data Rank	(R – Raverage)	(R – Raverage)2	(R – Raverage)3	(R – Raverage)4
1	59.80	(63.91)	4084.914178	- 261080.4815	16686523.84
2	66.00	(57.71)	3330.828844	- 192233.2354	11094420.79
3	72.00	(57.71)	2674.268844	- 138295.3562	7151713.852
4	88.50	(35.21)	1239.978844	- 43663.78838	1537547.535
5	88.50	(35.21)	1239.978844	- 43663.78838	1537547.535
6	89.60	(34.11)	1163.719511	- 39698.35159	1354243.101
7	91.60	(32.11)	1031.266178	- 3317.39452	1063509.929
8	95.00	(28.71)	824.4555111	- 23672.86591	679726.8898
9	101.20	(22.51)	506.8501778	- 11410.887	256897.1027
10	111.60	(12.11)	146.7328444	- 1777.423856	21530.52764
11	120.8	(2.91)	8.487511111	- 24.72694904	72.03784486
12	136.1	12.39	153.4295111	1900.480211	23540.61488
13	140.4	16.69	278.4448444	4646.316304	77531.5314
14	217.6	93.89	8814.706178	827583.3807	77699045
15	377	253.29	64154.13551	16249387.14	4115753103
Total	1,855.7		16242.99378	- 7888613.5727	41,383,661.10
Average	123.71				

From the above calculation, the appropriate type of probability distribution is determined. In determining the type of probability distribution, the following factors are needed[7]:

Table 5

Probability distribution calculation

Xaverage	=	123.71
Standard Deviation (S)	=	34.06
Variation Coefficient (Cv)	=	0.28
Slope Coefficient (Cs)	=	(1.64)
Sharpness Coefficient (Ck)	=	3.167253

In statistics, there are several types of distribution, among which are often used in hydrology are:

- Gumbel Distribution
- Log-Normal Distribution
- Log-Person type III distribution
- Normal Distribution

The following compares the distribution requirements and the analysis results of the frequency of rainfall calculation.

Table 6

Rainfall distribution calculation

No	Type of Distributions	Requirement	Calculation Result	Description
1	Gumbel	Cs < 1.1396 Ck < 5.4002	(1.64) 3.167252652	Eligible
2	Log Normal	Cs = 3 Cv + Cv2 Cs = 0.8325	(1.64) (1.64)	Not eligible
3	Log Pearson III	Cs ≈ 0	(1.64)	Not eligible
4	Normal	Cs ≈ 0	(1.64)	Not eligible

Based on calculations from the analysis of rainfall data, the type of distribution that meets the requirements is the Gumbel distribution.

Using the Gumbel Method, first look for the possibility of planned rainfall according to the return period of T years to determine the amount of planned flood discharge that will occur.

b. Determine S_n (Reduced Standard Deviation), Yn (reduced Mean), Reduced Variate (Y_t)

Table 7

S_n (Reduced Standard Deviation)

N	0	1	2	3	4	5	6	7	8	9
10	0.9496	0.9676	0.9833	0.9971	10.095	10.206	10.316	10.411	10.493	10.565
20	10.628	10.628	10.754	10.811	10.864	10.915	10.961	11.004	11.047	1.108
30	11.124	11.193	11.193	11.226	11.255	11.285	11.313	11.339	11.363	11.388
40	11.413	11.458	11.458	1.148	11.499	11.519	11.538	11.557	11.574	1.159
50	11.607	11.638	11.638	11.658	11.667	11.681	11.696	11.708	11.721	11.734
60	11.747	11.770	11.770	11.782	11.793	11.083	11.814	11.824	11.834	11.844
70	11.854	11.873	11.873	11.881	11.890	11.898	11.906	11.915	11.923	11.930
80	11.938	11.953	11.953	11.959	11.967	11.937	11.980	11.987	11.994	12.001
90	12.007	12.013	12.020	12.026	12.032	12.038	12.044	12.049	12.055	12.060
100	12.065	12.069	12.073	12.077	12.081	12.084	12.087	12.090	12.093	12.096

Source: Book of sustainable urban drainage systems

Table 8

Yn (reduced Mean)

N	0	1	2	3	4	5	6	7	8	9
10	0.4952	0.4996	0.5035	0.507	0.51	0.5128	0.5157	0.5181	0.5202	0.522
20	0.5236	0.5252	0.5268	0.5283	0.5296	0.5309	0.5332	0.5332	0.5343	0.5353
30	0.5362	0.5371	0.538	0.5388	0.8396	0.5403	0.5418	0.5418	0.5424	0.5436
40	0.5436	0.5442	15.448	0.2453	0.5458	0.5463	0.5468	0.5473	0.5477	0.5481

50	0.5485	0.5489	0.5493	0.5497	0.5501	0.5504	0.5508	0.5511	0.5515	0.5518
60	0.5521	0.5524	0.5527	0.5533	0.5533	0.5535	0.5538	0.5540	0.5543	0.5545
70	0.5548	0.5550	0.5552	0.5555	0.5557	0.5559	0.5561	0.5563	0.5565	0.5567
80	0.5569	0.5570	0.5572	0.5574	0.5576	0.5578	0.5580	0.5581	0.5583	0.5585
90	0.5586	0.5587	0.5589	0.5591	0.5592	0.5593	0.5595	0.5596	0.5598	0.5599
100	0.5600	0.5602	0.5603	0.5604	0.5606	0.5607	0.5608	0.5609	0.5610	0.5611

Source: Book of sustainable urban drainage systems

Table 9

Reduced Variate Table (Y_t)

Return Period	Reduced Variate
2	0.3665
5	1.4999
10	2.2502
20	2.9606
25	3.1985
50	3.9019
100	4.6001

Source: CD Soemarto, 1999

Analysis of the 5-year return period plan according to table 4.7. is YTr = 1.4999

After the S, Sn, YT r and Yn values are obtained, then proceed with analyzing the next maximum daily rainfall.

Table 10

Table of maximum rainfall calculation for the return period

Year	Sn	Yn	Sx	$X_{rata\text{-}rata}$	Ytr Table	(Yt-Yr)/Sn	Xtr
2	10.206	0.5128	34.06	123.71	0.3665	- 0.01433471	123.2250659
5	10.206	0.5128	34.06	123.71	1.4999	0.09671762	127.0077203
10	10.206	0.5128	34.06	123.71	2.2502	0.1702332	129.5118015
20	10.206	0.5128	34.06	123.71	2.9606	0.23983931	131.8827188
25	10.206	0.5128	34.06	123.71	3.1985	0.26314913	132.6766957
50	10.206	0.5128	34.06	123.71	3.9019	0.33206937	135.024251
100	10.206	0.5128	34.06	123.71	4.6001	0.40408011	137.3544515

c. Runoff Coefficient (C)

The catchment area is divided into three parts, namely buildings, parks, and roads, to determine the runoff coefficient (C)

Table 11

Runoff Coefficient Table (C)

No	Area	Size (A) Km2	Size (A) Km2	Coefficient		C
1	Building	186000	0.186	0.95	0.1767	
2	Garden area	62000	0.62	0.4	0.0248	
3	Road area	62000	0.62	0.95	0.0589	
			0.31		0.2604	0.84

d. Waterway Slope (S) dan Concentration Time (T)
The slope and length of the waterway are obtained by directly measuring using a total station tool. From the measurement results, the complex inlet has a length of 495.08 m, an upstream elevation of 23.98 m and a downstream elevation (PHB Pulo) of 21 m. Here is the slope calculation[8]:

$$S = \frac{\text{elevation A} - \text{elevation B}}{L} x100 \text{ .... } (14)$$

S = (23.98-21)/495.08 = 0.006

For calculation of concentration time (T_c):

$$t = 0.0195 \,.\, L^{0.77} \,.\, S^{-0.385} \text{ .... } (15)$$

$t = 0.0195 \times (495.08)^{0.77} \times (0.006)^{-0.385}$
= 1.628 hours

e. Plan rainfall intensity (I)

The available rain data is the average daily maximum rain data so that the calculation of rainfall intensity using the formula from Monobe[9]. The duration of the rain is assumed to be the same as the concentration-time value obtained in the previous total.
The results of calculations for other return periods can be seen in the following table:

Table 12

Rainfall Intensity Table (I)

No	Return Period	Plan Rainfall	tc	Rainfall Intensity
1	2	123.2250659	1.628022	30.89664941
2	5	127.0077203	1.628022	31.84508751
3	10	129.5118015	1.628022	32.47294449
4	20	131.8827188	1.628022	33.06741284
5	25	132.6766957	1.628022	33.26648945
6	50	135.024251	1.628022	33.85510014
7	100	137.3544515	1.628022	34.43935943

The 5-year return period (T) value is taken with an intensity value of 31.845 mm/hour.

f. Plan waterway debit ($Q_{hydrology}$)

The design debit equation ($Q_{hydrology}$) can be calculated using the rational method formula.

Q = 0,278.C.I.A

Table 13

Design waterway debit calculation ($Q_{Hydrology}$)

No	Return Period	Plan Rainfall	C	Catchment Area (km2)	Debit m3/s
1	2	30.89664941	0.84	0.31	2.236646
2	5	31.84508751	0.84	0.31	2.305304
3	10	32.47294449	0.84	0.31	2.350755
4	20	33.06741284	0.84	0.31	2.39379

5	25	33.26648945	0.84	0.31	2.408201
6	50	33.85510014	0.84	0.31	2.450811
7	100	34.43935943	0.84	0.31	2.493107

Existing Waterway Debiy ($Q_{Hydraulic}$)

For the Manning coefficient value (n), because the channel wall construction is made of concrete/U-ditch, the coefficient value is 0.014[10]. The following is a table of the results of the calculation of the existing waterway debit ($Q_{Hidrolika}$) (the dimensions of the waterway are attached)

Table 14

The calculation of the waterway debit plan ($Q_{Hydraulic}$)

Location	b	h	A	P	R=A/P	S	n	R2/3	S1/2	V=1/nt . R2/3 . S1/2	Q=At . V	0,5 th	Description
Waterways	1	1	1	3	0.333333333	0.60%	0.024	0.4807	0.0776	2.6642	2.6642	2.305	Appropriate

Description:
b = width waterway (m)
h= height waterway (m)
A = Cross sectional area (m^2)
P = Wet cross section (m)
R= Cross sectional area/wet cross section
S= Slope
n = Roughness coefficient
V= Velocity (m/s)
Q= Debit (m^3/s)

Pump Capacity Requirements

From the analysis of the planned debit, the intensity of rainfall is generated. It is then analyzed for pump capacity requirements in the complex area as follows:

a. Estimated Treated Pond Volume
V= b x h x waterway length
= 1 x 1 x 495.08 meter = 495.08 m^3
b. Pond Storage Volume
V= length x width x height
= 15 x 8 x 6
= 1,215.08 m^3
c. Runoff coefficient = 0.84
d. Rainfall Intensity = 31.845 mm/hour
e. Drain Time = 2 hours
f. Area = 0.31 km^2

Using the formula for the volume of water on the surface
V = 1/3.6 x C x R x A x T
V = 1/3.6 x 0.84 x 31,845 x 0.31 x 3600
= 8,292.46 m^3

The volume of water to be pumped = (Volume of surface water - Drainage capacity - Treatment pond holding capacity)/3600 for 2 hours.

Then the volume to be pumped = (8,292.46 m^3 – 495.08 m^3 – 1,215.08 m^3)/(2 x 3,600) = 0.98 m^3/second, rounded to 1 m^3/second.

CONCLUSIONS

Based on the results of data analysis, several conclusions are obtained, including:

1. Design waterway debit (Qhydrology) for five year return period is 2.305 m3/second.
2. A complex area waterway requires a channel cross-section of at least 1 x 1 m2 to accommodate a 2.664 m3/second waterway debit using U-ditch or concrete construction. The water level of PHB Pulo causes queues in the waterway so that a pump of 1 m3/second is needed.

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