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Evaluation of Drainage System on Public Road in Front of Sidoarjo Religious Court Office Class 1A

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ABSTRACT

Flooding in front of the Sidoarjo Class 1A Religious Court Office road happens frequently during the rainy season. It disrupts traffic and damages the roads. Flooding is caused by the condition of the drainage channels, which are not performing efficiently due to sediment barriers such as garbage, and some channels appear to be damaged. The existing channels can no longer accommodate the water discharge caused by the rain, thus water overflows from the drainage channels, flooding the main road. These conditions show the importance of handling drainage problems. One of the ways that can be taken is to evaluate the capacity of existing drainage channels.In this study, the evaluation of drainage channel capacity was carried out using the Hydrological and Hydraulics Analysis method. The purpose of this study was to determine the amount of drainage channel storage capacity required to resolve flooding. The results of this study showed that the design rainfall obtained based on the Log Pearson Type III method was 116.583 mm/day with a 5-year return period. The design flood discharge in channel T1 was 0.717 m3/second, while the capacity of the existing tertiary channel T1 = 0.009 m3/second, so the discharge that overflows from the existing channel T1 was 0.708 m3/second. The capacity of the secondary channel S1 was 2.604 m3 /second, whereas the capacity of the primary channel P1 was 43.043 m3 /second. The capacity of the T1 channel was known to be unable to accommodate the design flood discharge. Therefore, alternative treatments were required to ensure that the T1 channel could accommodate the designed flood discharge.

Keywords: Drainage, Flood, Hydrological, Hydraulic

1. Introduction

The Religious Court of Sidoarjo Class 1A is a legal institution that plays a central role in resolving civil and family cases in the area. Situated in a strategic location, this office serves as a crucial legal enforcement center for the community. Wijaya Kusuma Street in front of the office is not only a public traffic route but also the main access for residents to engage in various daily activities, including accessing legal services, government services, and other social activities. Good connectivity through public roads supports the smooth and effective movement of visitors, judges, employees, and other stakeholders who require smooth and safe access to the Religious Court of Sidoarjo Class 1A. Therefore, the smooth flow of public roads around this office directly impacts the efficiency of community activities as a whole.

Floods in front of the Sidoarjo Religious Court Class 1A often occur when the rainy season comes. This condition causes traffic disruptions and damages to the road surface. The impact of floods on road performance includes significant traffic disruptions due to water puddles, resulting in traffic congestion and travel delays for road users. In addition, floods can also cause damage to road infrastructure, such as damaged roads and clogged drainage systems, all of which affect the quality of the road and the comfort of road users.

This infrastructure damage can also affect accessibility, limiting the movement of the community and increasing the risk of accidents on the highway due to dangerous road conditions.

The flood that occurred in the public road area in front of the Sidoarjo Class 1A Religious Court Office was caused by the malfunctioning of the drainage system, due to the presence of sediment and debris in the channels, as well as some damaged channels. The existing channels were no longer able to accommodate the water flow from the rain, resulting in the main road being flooded (Yulius, 2018).

This condition highlights the importance of addressing the drainage problem in the public road area in front of the Sidoarjo Class 1A Religious Court Office. One of the ways to address the flooding and waterlogging issues is by evaluating the capacity of each existing drainage channel. The flood and waterlogging problems in the public road area in front of the Sidoarjo Class 1A Religious Court Office can be overcome by redesigning the drainage channels according to the flood capacity in each segment of the channels (Guntoro et al., 2017).

In this study, the evaluation of drainage capacity is conducted using Hydrological and Hydraulic Analysis methods. The aim of this study is to determine the storage capacity of the drainage channels, so that floods can be mitigated, analyze the suitability of the flood discharge for a 5-year return period with the existing drainage channel capacity, and find solutions if the drainage system is no longer able to accommodate the planned flood discharge.

2. Literature Review

Drainage comes from English drainage, meaning to drain, dispose of, or divert. In the field of civil engineering, drainage is generally used to reduce excess water such as rainwater, seepage water, or excess irrigation water from an area/land, so that the function of the area/land is not disturbed (Juliana, 2019). In general, a drainage system can be defined as a set of water structures designed to reduce and or drain excess water from an area/land so that land utilization can be carried out optimally (Suripin, 2004).

A drainage system also can be defined as a series of water structures that function to reduce and/or remove excess water from an area/land, so that the land can be optimally utilized (Lukman, 2018). Drainage system buildings sequentially starting from upstream consist of interceptor drains, collector drains, conveyor drains, main drains, and receiving waters (Sultonulazkar et al., 2022). Drainage system buildings that are often found, such as culverts, bridges, gutters and sloping channels (Andana et al., 2016). Sediment in a river commonly mentioned as increasing the risk of flooding due (Patriadi et al., 2022).

2.1. Hydrological Analysis

Hydrological analysis is an initial analysis related to the design of flood management systems and drainage system planning, determining the amount of runoff that will be carried so that the dimensions of the drainage channel can be determined (Linsley Jr et al., 1975). The flow rate used as a basic criterion in drainage planning, especially flood management, is the design flow rate, which is the amount of rain runoff planned for a certain return period and wastewater discharge from an area (Krisnayanti et al., 2017). The analysis is carried out by calculating the average rainfall of the flow area using the Thiessen Polygon method. To calculate the annual rainfall plan can use the Normal Method, Log Normal, Gumbel, Log Pearson III Distribution (Asmorowati et al., 2021).

a. Calculation of Flow Coefficient (C)

Surface runoff time is the time it takes to runoff rainwater from the farthest point to the nearest channel, often referred to as inlet time, overflow time (t_0) in minutes. The formula for calculating t_0 is:

$$t_0 = \left(\frac{2}{3} . 3{,}28 . L_o \frac{n_d}{\sqrt{S}}\right)^{0.167}$$

Flow time is the time required to drain water in the channel, from one point of entry of runoff water to the review point. The amount of flow time according to SNI on Planning Procedures for Road Surface Drainage: 1994, is:

$$t_d = \frac{Ld}{60.Vsal}$$

The velocity of water in the channel depends on the material of which the channel is made. Concentration time or flood arrival time is the time required by rainwater to flow from the most distant point to a certain review point (e.g. point at the mouth of the drainage) in a drainage area (Rachman et al., 2014).

$$T_c = t_0 + t_d$$

b. Calculation of Rain Intensity (I)

If only daily rainfall data is available, the rainfall intensity can be calculated using the Mononobe formula. Calculation of design rainfall intensity is done based on the Mononobe Method:

$$I = \frac{R24}{24} X \left(\frac{24}{t}\right)^{2/3}$$

c. Flood Discharge Calculation (Q)

A commonly used method for urban drainage, estimating peak flow rates (plan discharge), is the Rational Method. The rational method is developed based on the assumption that rainfall that occurs has a uniform intensity and is evenly distributed throughout the drainage area for at least equal to the time of concentration (tc). Channel dimensions are planned based on the amount of rainwater discharge that will be flowed using the Rational Method The mathematical equation of the Rational Method is as follows:

$$Q = 0{,}278 \,.\, C \,.\, I \,.\, A$$

2.2. Hydraulics Analysis

Hydraulics analysis is intended to find the hydraulic dimensions of the drainage channel and its complementary buildings. In determining the dimensions of the drainage channel, it is necessary to take into account the planning criteria based on the rules of hydraulics. Channel Capacity In the early stages of the analysis it is assumed that what occurs is uniform flow (Mahendra et al., 2023). Analysis to calculate channel capacity, the continuity equation and Manning's formula are used, namely:

$$R = \frac{A}{P}$$

In this case, the values of A and P are obtained from the following equations:

A = (B + mh) H (for trapezoidal cross-section channels)

A = B x H (for square section channels)

P = B + 2H + √1+m^2 (for trapezoidal cross-section channels)

P = B + 2H (for square section channels)

Where the value of m is obtained from:

$$m = \frac{B - b}{2 X H}$$

Then calculate the channel capacity discharge with the following equation:

$$Q = V \,.\, A$$

Where the value of V is obtained from the following equation:

$$V = \frac{1}{n} x R^{2/3} x S^{1/2}$$

$$S = \left(\frac{v x n}{R^{2/3}}\right)^2$$

3. Methodology

This research is located on a public road in front of the Sidoarjo Class 1A Religious Court Office, namely Wijaya Kusuma Street. The stages carried out in this research are presented in a flowchart according to Figure 1.

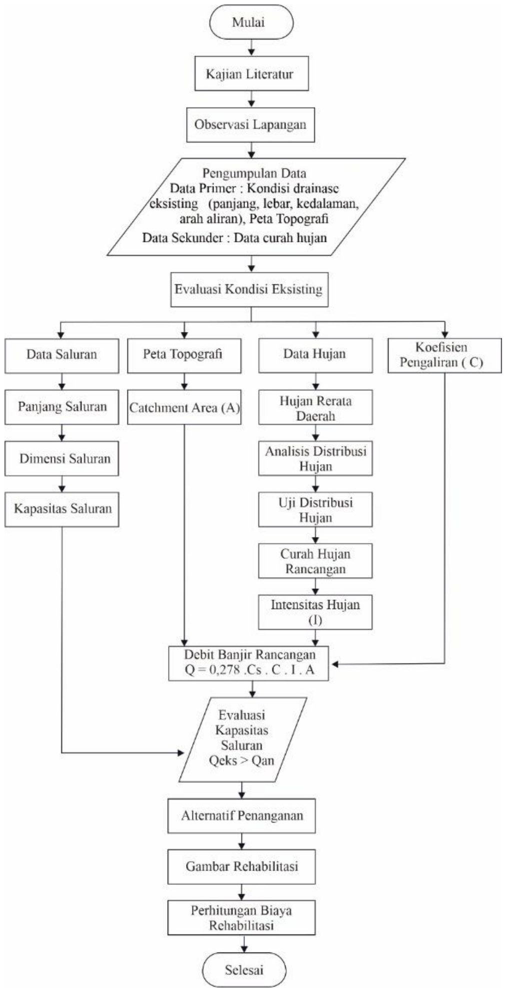

Figure 1. Research Stages

3.1. Research Data

a. Primary data

Survey of existing channel conditions in the form of measuring channel dimensions, length, width, depth and flow direction contained in the channel. To measure the height of sediment in the channel manually, namely using wood, then plugged until it touches the bottom of the channel, then the value of the sediment height can be known (Agustyawan & Arif, 2020). Making topographic maps with Google Earth Pro and ArcMap software according to the delineation of the area to be studied (Yu & Peng, 2012).

b. Secondary data

Secondary data, was obtained through literature studies and collecting data and information from various sources. The secondary data in this study are rainfall data from 2011-2023 (13 years).

3.2. Data Analysis Technique

Data analysis activities are carried out using Hydrological analysis and Hydraulics analysis methods. The stages of data analysis carried out from the basic data include:

a. Analyze hydrological data, to obtain planned rainfall and design discharge for existing and planning evaluation (Azhari, 2021).

b. Analyzing hydraulics data, such as analyzing channel capacity after obtaining existing channel discharge
c. Evaluation of channel capacity (Widiastomo et al., 2022).
d. Calculating the rehabilitation cost budget plan.

4. Results and Discussion

4.1. Hydrological Analysis

This analysis aims to determine the amount of plan flood discharge with a certain return period (Balahanti et al., 2023). The data used in this calculation is rainfall data. The first step in the analysis of daily average rainfall is to determine the location of the rain station used. After analyzing the Thiessen Polygon Method by connecting three rain stations and then cutting perpendicularly in the middle of the axis, one rain station that affects the drainage system of Jalan Wijaya Kusuma is obtained, namely the Sidoarjo Rain Station (Astika & Okik, 2020). Can be seen in Figure 2.

Figure 2. Wijaya Kusuma Road Drainage System, Sidoarjo Rain Station

The rainfall data used is the maximum daily rainfall data for the last 13 years, from 2011 to 2023.

Table 1. Maximum Daily Rainfall Data for 2011-2023

No.	Years	SH Sidoarjo
		mm
1.	2011	113
2.	2012	84
3.	2013	96
4.	2014	76.5
5.	2015	96
6.	2016	170
7.	2017	110
8.	2018	109
9.	2019	97
10.	2020	98
11.	2021	101
12.	2022	101
13.	2023	72

From the annual maximum rainfall data calculate the design rainfall with several methods such as Gumbel, Normal, and Log Pearson Type III. In determining the method based on the value of Skewness (Cs) and Kurtosis coefficient (Ck).

Table 2. Calculation of Design Rainfall Based on Annual Maximum Rainfall Data with Gumbel, Normal, and Log Pearson Type III Methods

No	Year	Xi	$(Xi - \bar{X})$	$(Xi - \bar{X})^2$	$(Xi - \bar{X})^3$	$(Xi - \bar{X})^4$
1	2011	113	11,22	125,90	1412,60	15849,85
2	2012	84	-17,78	316,12	-5620,43	99929,34
3	2013	96	-5,78	33,40	-193,07	1115,85
4	2014	77	-25,28	639,06	-16155,23	408398,72
5	2015	96	-6,14	37,75	-231,95	1425,13
6	2016	170	68,22	4654,02	317498,58	21659863,69
7	2017	110	8,22	67,57	555,48	4566,26
8	2018	109	7,22	52,13	376,42	2717,89
9	2019	97	-4,78	22,85	-109,19	521,90
10	2020	98	-3,78	14,29	-54,00	204,08
11	2021	101	-0,78	0,61	-0,47	0,37
12	2022	101	-0,78	0,61	-0,47	0,37
13	2023	72	-29,78	886,83	-26409,42	786463,33
Total		1323	0	6851,13	271068,85	22981056,80
$\bar{X}$		101,78				
Total Data (n)		13				
Standard Deviation (S)		23,89				
Skewness Value (Cs)		1,96				
Coefficient Kurtosis (Ck)		9,03				

From the calculation of statistical parameters, the coefficient of skewness (Cs), and the coefficient of sharpness (Ck) will be checked with the existing conditions as shown in Table 3.

Table 3. Statistical Parameter Checking to Determine Distribution Type

No.	Distribution Type	Parameter	Result	Description
1.	Normal	$Cs \approx 0$	Cs = 1,96	does not meet
		$Ck \approx 3$	Ck = 9,03	does not meet
2.	Log Normal	$Cs \approx 0$	Cs = 1,96	does not meet
		$Ck > 3$	Ck = 9,03	does not meet
3.	Gumbel	$Cs \leq 1,1396$	Cs = 1,96	does not meet

No.	Distribution Type	Parameter	Result	Description
		Ck ≤ 5,4002	Ck = 9,03	does not meet
4.	*Log Pearson Tipe III*	Ck= Fleksibel	Cs = 1,96	meet
		Ck = Fleksibel	Ck = 9,03	meet

Based on the calculation results of Table 3, the value of Cs is 1.96 and Ck is 9.03. It can be concluded that the appropriate distribution according to the applicable requirements is the Log Pearson Type III Distribution. Furthermore, the distribution suitability test is carried out, the purpose of the suitability test is to check whether the appropriate type of distribution can be used for further calculations.

Average rainfall value, standard deviation price, Cs price, Ck price, and Ck price in the calculation of the Log Pearson Type III method.

Table 4. Rainfall Data Statistical Analysis Table

No	**Xi**	**Log Xi**	**(Log Xi - Log $\bar{X}$)**	**(Log Xi - Log $\bar{X}$)^2**	**(Log Xi - Log $\bar{X}$)^3**
[1]	[2]	[3]	[4]	[5]	[6]
1	170	2,230	0,232	0,054	0,013
2	113	2,053	0,055	0,003	0,000
3	110	2,041	0,043	0,002	0,000
4	109	2,037	0,039	0,002	0,000
5	101	2,004	0,006	0,000	0,000
6	101	2,004	0,006	0,000	0,000
7	98	1,991	-0,007	0,000	0,000
8	97	1,987	-0,011	0,000	0,000
9	96	1,982	-0,016	0,000	0,000
10	96	1,981	-0,018	0,000	0,000
11	84	1,924	-0,074	0,005	0,000
12	77	1,884	-0,115	0,013	-0,002
13	72	1,857	-0,141	0,020	-0,003
Total (ΣXi)		**25,98**	**-0,33**	**0,04**	**0,00**
Average (Log $\bar{X}$)		**1,998**			
Standard Deviation (S)		**0,091**			
Skewness Value (Cs)		**1,056**			

Table 5. K Values at Various Rates of Return for the Log Pearson Type III Distribution

Years	1,0	1,25	2	5	10	25	50	100
Percent	**99,0**	**80**	**50**	**20**	**10**	**4**	**2**	**1**
1,0	-1,588	-0,852	-0,164	0,758	1,340	2,043	2,542	3,022
1,2	-1,449	-0,844	-0,195	0,732	1,340	2,087	2,626	3,149
1,056	-1,549	-0,850	-0,173	0,751	1,340	2,055	2,566	3,058

Table 6. Calculation Results of XT Value Using Log Pearson Type III Method

T (Yrs)	Opportunities (%)	G	Log XT (mm)	XT (mm)
[1]	**[2]**	**[3]**	**[4]**	**[5]**
5	20	0,751	2,067	116,58

After conducting the rainfall frequency analysis, parameter testing is required to determine the goodness of fit of the distribution of the sample data to the probability distribution function that is expected to describe/represent the frequency distribution. The fit test is intended to determine whether each probability distribution can represent the statistical distribution of the data samples being analyzed. The parameter tests used are the Chi-Square test and the Smirnov-Kolmogorov test.

Table 7. Fit Test Testing Parameters for the Log Pearson Type II DistributionI

m	Xi (mm/days)	log Xi	P = m/(n+1)
1	170	2,23	12,14
2	113	2,05	8,07
3	110	2,04	7,86
4	109	2,04	7,79
5	101	2,00	7,21
6	101	2,00	7,21
7	98	1,99	7,00
8	97	1,99	6,93
9	96	1,98	6,86
10	96	1,98	6,83
11	84	1,92	6,00
12	77	1,88	5,46
13	72	1,86	5,14
Number of Data (n)		**13**	
Average (Log $\bar{X}$)		**1,998**	
Cs		**1,056**	
S (standard deviation)		**0,091**	

Recapitulation table of G value interpolation calculation results for Cs between 1 and 1.2

Table 8. Recapitulation of G Value Interpolation Calculation Results for Cs between 1 and 1.2

Years	**1,25**	**1,67**	**2**	**2,5**	**5**
Percent	**80**	**60**	**50**	**40**	**20**
1,00	-0,852	-0,393	-0,164	0,143	0,758
1,20	-0,844	-0,411	-0,195	0,114	0,732
1,056	**-0,850**	-0,398	**-0,173**	0,135	**0,751**

Table 9. Calculation of T Value with Probability and Results of Regional Average Rainfall Analysis

No	**P (%)**	**T = 1 / P (Year)**	**log $\bar{X}$**	**G**	**$S_{\log X}$**	**log X_T**	**X_T (mm/day)**
1	20	5,00	1,998	0,7507	0,091	2,0666	116,58
2	40	2,50	1,998	0,1351	0,091	2,0106	102,46
3	60	1,67	1,998	-0,3984	0,091	1,9619	91,61
4	80	1,25	1,998	-0,8498	0,091	1,9208	83,33

By sorting the results of the regional average rainfall analysis starting from the largest value to the smallest, the following is the calculation of the Chi-Square test for the Log Pearson Type III Distribution.

Table 10. Calculation of Chi-Square Test for Log Pearson Type III Distribution Based on Rainfall Analysis

No	**Class Limits**			**Total Data**		**O_i - E_i**	**$(O_i - E_i)^2/ E_i$**
				O_i	**E_i**		
1	0	P ≤	83,33	2	2,6	-0,60	0,14
2	83,33	< P ≤	91,61	1	2,6	-1,60	0,98
3	91,61	< P ≤	102,46	6	2,6	3,40	4,45
4	102,46	< P ≤	116,58	3	2,6	0,40	0,06
5	116,58	< P ≤	P	1	2,6	-1,60	0,98
	Total			13	13		6,62

Count Value X^2	6,615	
Table Value X^2	7,815	**Accepted**

The value of d-count is 6.615 according to the probability table with the value of dk and percent chance, the critical dcount for 5% is 7.815. Since d-count < d-table, the test for Log Pearson Type III Distribution is accepted.

Table 11. Calculation of Probability and Cumulative Frequency for Log Pearson Type III Distribution

m	X	Log X	P (Log X)	P (Log X<)	f(t)	P' (Log X<)	P' (Log X)	D
[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]
	mm	log [X]	m/[n+1]	1-P (logX)	(Log X - Log X̅)/ SlogX	Table	1-P' (Log X<)	[4]-[8]
1	170	2,230	0,071	0,929	2,55	0,995	0,005	0,066
2	113	2,053	0,143	0,857	0,60	0,726	0,274	0,131
3	110	2,041	0,214	0,786	0,47	0,681	0,319	0,105
4	109	2,037	0,286	0,714	0,43	0,666	0,334	0,048
5	101	2,004	0,357	0,643	0,07	0,528	0,472	0,115
6	101	2,004	0,429	0,571	0,07	0,528	0,472	0,044
7	98	1,991	0,500	0,500	-0,08	0,468	0,532	0,032
8	97	1,987	0,571	0,429	-0,13	0,448	0,552	0,020
9	96	1,982	0,643	0,357	-0,18	0,429	0,571	0,071
10	96	1,981	0,714	0,286	-0,19	0,425	0,575	0,139
11	84	1,924	0,786	0,214	-0,81	0,209	0,791	0,005
12	77	1,884	0,857	0,143	-1,26	0,104	0,896	0,039
13	72	1,857	0,929	0,071	-1,55	0,061	0,939	0,011

X̅		1,998	
Sd		0,091	
Cs		1,056	
Dmax		0,139	
D0 table	5%	0,361	**Accepted**

The value of d-count is 0.139, according to the probability table with the value of dk and percent chance, the critical dcount for 5% is 0.361. Because d-count < d-table, the test for the Log Pearson Distribution is accepted.

The planned flood discharge is the flood discharge used as the basis for planning the level of flood hazard observation at the observation location by applying the value of the probability of the largest flood. In the calculation of this plan flood discharge analysis using the calculation of Q with a return period of 5 years

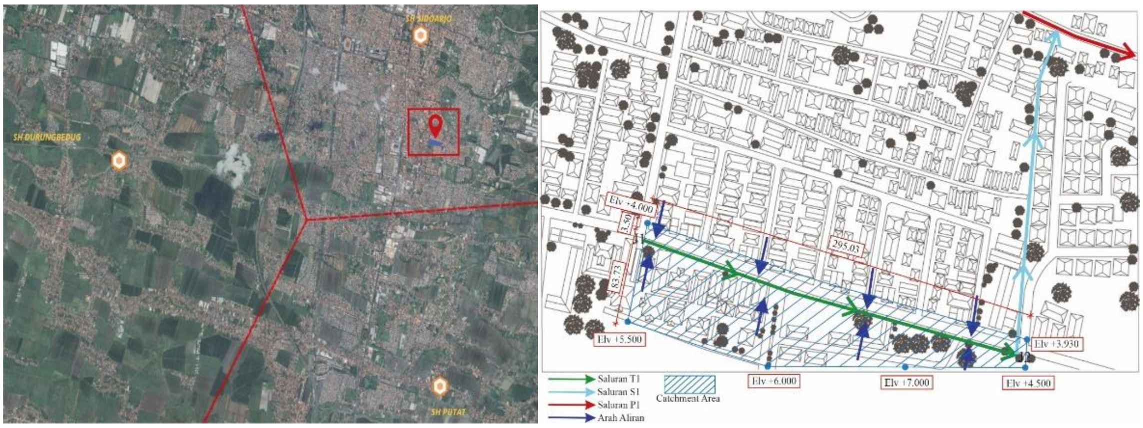

Figure 2. Planned Flood Discharge

Table 12. Flood Discharge Analysis Calculation Using Q Calculation with a Return Period of 5 years

Point	Channel Name	Channel Type	Source	Area (m²)	C	Cmerged
J1 J2	T1	Tertiary	Road	1.032,5	0,95	0,52
			Cemetery	22.750	0,5	

The calculation of flow time consists of the time of water flow on the land surface entering the channel (t_0), the calculation of the time of water flowing along the channel (t_d), and the concentration time or time required by the water point to flow from the farthest hydraulic place in the flow area to a point under review (t_c).

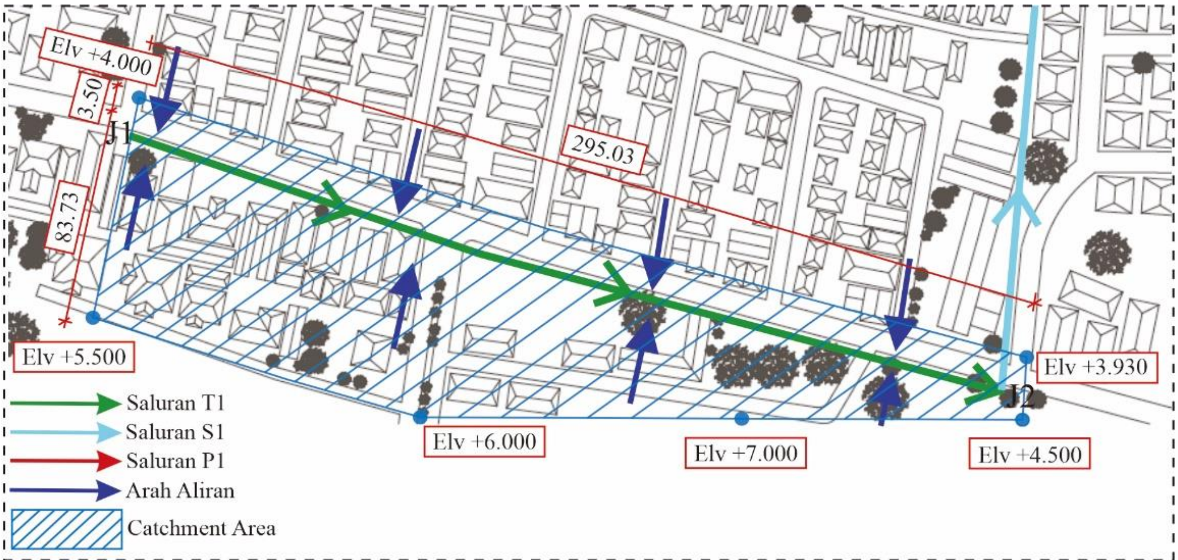

Figure 3. The Flow Area To a Point Under Review

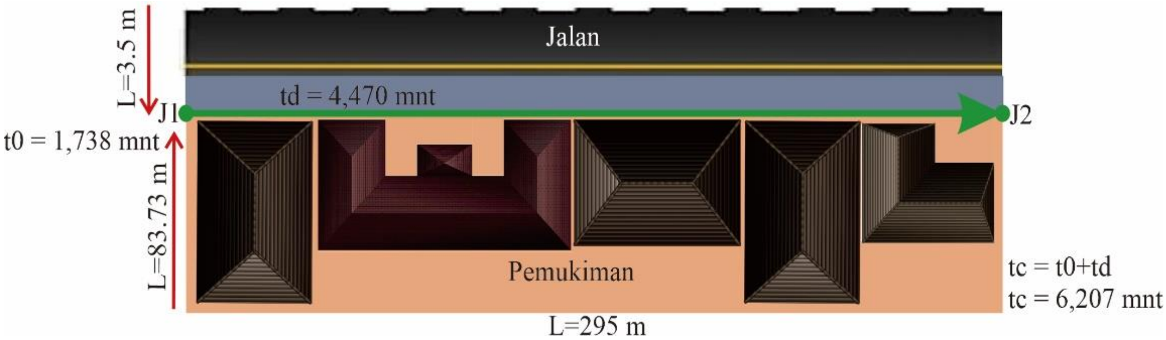

Figure 4. Road and Settlement Area

Table 13. Channel Parameters, Channel Flow Data, and Total Flow Time

Point	Channel	Drainage	L_0	N_0	S_0	T_0	T_{0max}
			m		1%-2%	minutes	minutes
J1-J2	T1	Road	3,50	0,013	2,0%	0,943	1,824
		Residential	83,73	0,020	1%	1,824	
Point	**Channel**	**Drainage**	Ld	Vd	Td		
			m	m/s	min		
J1-J2	T1	Road	295	1,5	3,278		
		Residential	295	1,5	3,278		
Point	**Channel**	**Drainage**	$t_0$max	Td	Tc		
			min	min	min	hours	
J1-J2	T1	J1-J2	1,824	3,278	5,102	0,085	

Table 14. Rainfall intensity is calculated with the Mononobe Method

Point	Channel	t_c		R5	I
		min	Hours	mm	mm/Hours
J1-J2	T1	6,207	0,085	116,583	209,006

From the results of previous calculations, supporting data were obtained to calculate the design flood discharge using the Rational method, namely the value of the conveyance coefficient (C), rainfall intensity (I), and catchment area (A). With the formula of the Rational method, the design flood calculation is as follows:

Table 15. Design Flood Discharge Using Rational Method

Point	Channel	I	C	A	Q5
		mm/hours		km^2	m^3/s
J1-J2	T1	209,006	0,52	0,0238	0,717

4.2. Hydraulics Analysis

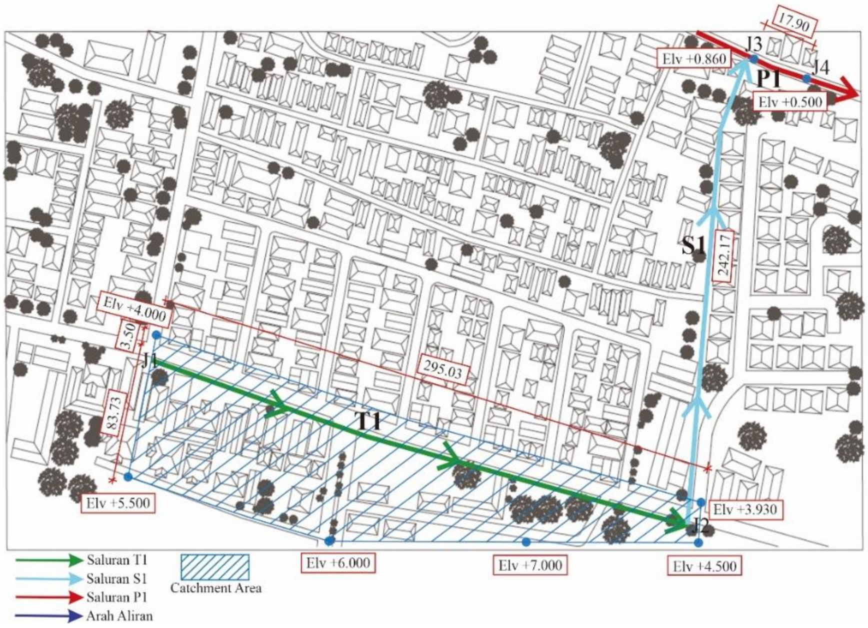

Figure 5. Hydraulics Channel Analysis

Based the observation, it was found that on Jl Wijaya Kusuma, the channel only exists on the right side of the road, and there was inundation at the location of the three intersection in front of the Sidoarjo Class 1A Religious Court Office along 75.85 m (Savitri, 2017). On the right side of the channel, the existing dimensions of the channel are known to be 0.3 m wide and 0.3 m deep, but because there is sedimentation along the channel, the effective water space is only 0.20 m high (Akan & Iyer, 2021). To support the results of these observations must be supported by the results of hydraulics calculations as a basis for evaluating channel performance as follows:

Table 16. Hydraulics Calculation Results

Point	Ld	Original Ground Level Elevation			Channel cross-section		n
	m	Initial	Final	Original S	Shape	Material	
J1-J2	295	+4,000	+3,930	0,02%	Square	Concrete	0,019

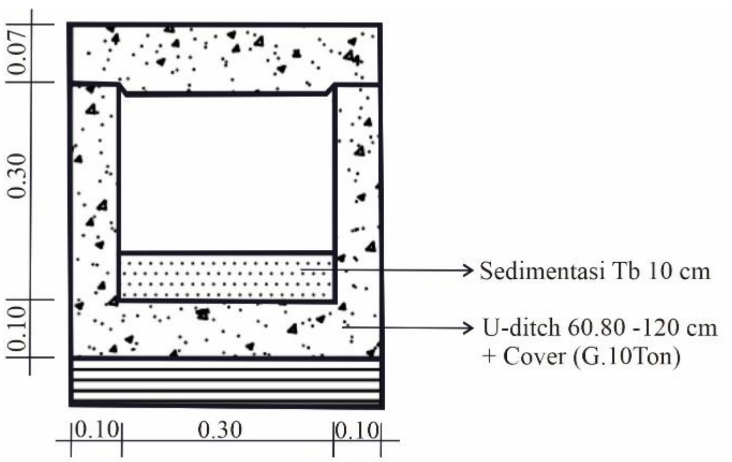

Figure 6. Sedimentation Details

Table 17. Results of Calculation of Capacity that can be Accommodated by Existing Channels

B	H	A	P	R	V	Q hydraulics for existing struc.
(m)	(m)	(m^2)	(m)	(m)	(m/s)	(m^3/s)
0,30	0,20	0,060	0,700	0,086	0,158	0,009

It is known that the capacity that can be accommodated by the existing channel is 0.009 m^3 / sec. The indicator that the channel can accommodate flood discharge is if the channel Qhydraulic is equal to or greater than the planned flood discharge, the comparison of the existing channel discharge (Qhydraulic) with the design flood discharge (Q5). The runoff that occurs in the public road channel in front of the Sidoarjo Class 1A Religious Court Office is 0.704 m^3 / second (Sulistiono & Ardiyanto, 2016).

Table 18. Existing Channel Capacity

Q5 (m^3/ sec)	Qhydraulic (m^3/ sec)	Qlimpasan (m^3/ sec)	Conclusion
0,713	0,009	0,708	The existing channel cannot accommodate the planned flood discharge with a 5-year return period

Table 19. Channel Cross Section Detail Results

Point	Ld	Original Ground Level Elevation			Channel cross-section		n
	m	Initial	Final	S real	Shape	Material	
J2 - J3	243	+3,930	+0,860	1,27%	Square	Concrete	0,019

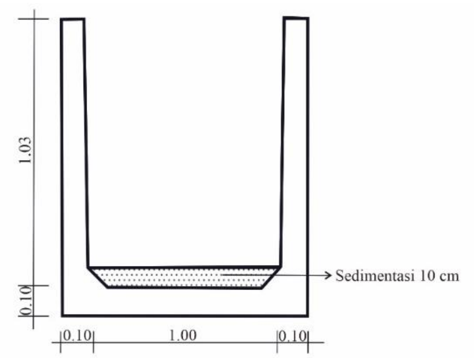

Figure 7. Channel Cross Section J2-J3

Table 20. Secondary Channel Hydraulics Details

B	H	A	P	R	Veks	Qhydraulic of Secondary Channel Struc.
(m)	(m)	(m^2)	(m)	(m)	(m/sec	(m^3/sec)
1	0,93	0,930	2,860	0,325	2,800	2,604

From the results of the above calculations, it is obtained that the capacity that can be accommodated by the existing S1 channel is 2.604 m3 / sec. The indicator that the channel can accommodate flood discharge is if the Qhydraulic channel is equal to or greater than the planned flood discharge, the comparison of the existing channel discharge (Qhydraulic) with the design flood discharge (Q5) (Yanti et al., 2019).

Table 21. Comparison of Existing Channel Discharge with Plan Flood Discharge

Q5 (m^3/sec)	Qhydraulic (m^3/sec)	Qlimpasan (m^3/sec)	Conclusion
0,713	2,604	-	Secondary Channel S1 can still accommodate the design flood discharge. Qhydraulics > Q5

Table 22. Channel Details and Characteristics

Point	Channel	Elevation		Ld	n
J3 - J4	P1	+0,860	0,500	17,90	0,033

The length of the channel identified in this study is 18.16 m long. In the table above, the manning value used is 0.033 where the channel is made of stone masonry with moderate conditions as listed in the Manning roughness table according to the type and condition of the channel (Road Drainage Design Guidelines, 2021).

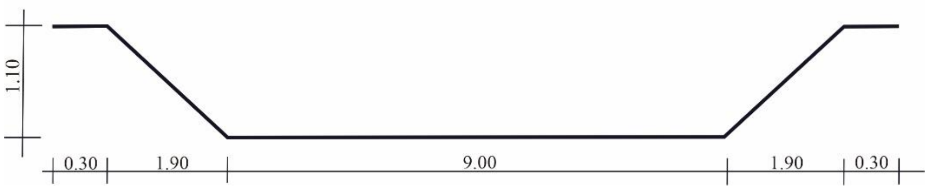

Figure 8. Road Drainage Design Guidelines, 2021

Table 23. Comparison of Existing Channel Discharge with Design Flood Discharge

Existing Channel Dimensions									
m	b	B	H	A	P	R	S	V	Q
	(m)	(m)	(m)	(m^2)	(m)	(m)		(m/sec)	(m^3/sec)
0,58	9	12,9	1,10	10,6	11,5	0,92	2,01%	4,06	43,043

From the results of the above calculations, it is obtained that the capacity that can be accommodated by the P1 channel is 43.04 m3 / second The indicator that the channel can accommodate flood discharge is if the channel Qhydraulic is equal to or greater than the planned flood discharge, the comparison of the existing channel discharge (Qhydraulic) with the design flood discharge (Q5).

Table 24. Comparison of Existing Channel Discharge (Qhydraulics) with Design Flood Discharge (Q5)

Q5 (m^3/sec)	Qhydraulic (m^3/sec)	Qlimpasan (m^3/sec)	Conclusion
0,717	43,043	-	Primary Channel P1 can still accommodate the design flood discharge. Qhydraulics > Q5

5. Conclusion

Based on the calculation results, the following conclusions can be drawn:

1. Based on the design rainfall obtained using the Log Pearson Type III method, it is 116.583 mm/day with a return period of 5 years.
2. The design flood discharge for channel T1 is 0.717 m^3/second, while the capacity of the existing channel T1 is 0.009 m^3/second, resulting in an overflow discharge of 0.708 m^3/second from the existing channel T1. The capacity of secondary channel S1 is 2.604 m^3/second, and the capacity of primary channel P1 is 43.043 m^3/second. It is known that the capacity of channel T1 cannot accommodate the design flood discharge; therefore, alternative measures are required to ensure that channel T1 can manage the design flood discharge.

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