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Evaluation of Flood Control Performance in Talangsari Watershed, Samarinda, East Kalimantan

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ABSTRACT

The flood issues in the Talangsari River in Samarinda City were not only caused by high rainfall but also by the significant sediment accumulation and waste along the river, the encroachment of residential areas onto the river section, and the lack of public awareness regarding environmental cleanliness. As a result, flooding occurs throughout the Talangsari River, from upstream to downstream, indicating significant damage to the river. Considering these issues, proactive measures are necessary to address the flood problems. One such measure is the normalization of the Talangsari River to ensure smooth land transportation from and to Apt Pranoto Airport in the Sei Siring area, Bontang City, Sangatta, and Kutai Kartanegara. The flood discharge plan was estimated using Nakayasu and SCS synthetic unit hydrograph method. The analysis revealed a peak discharge of 6.39 m^3/s for the existing conditions (2-year return period) and 13.38 m^3/s for the normalization conditions (25-year return period). Subsequently, the simulations were performed using the HEC-RAS program by two scenarios: the existing river condition and the normalization scenarios. Following the simulation with the 2-year return period discharge for the existing conditions, the Talangsari River experienced upstream to downstream flooding, underscoring the need for flood mitigation measures. The river normalization simulation using the 25-year return period discharge considered dimensions such as a channel width of 7m and a depth of 3.5m. As a result of the normalization simulation, the Talangsari River successfully accommodated a flow of 13.38 m^3/s without experiencing flooding, confirming that the decision to pursue normalization was the best choice.

1. Introduction

A river is a natural or artificial watercourse and container consisting of a flowing water network, starting from the source, and ending at the mouth, bounded on the right and left by its banks [1]. Over time, rivers have become more than just a water source; they have become essential to communities in meeting their daily needs.

Like the Talangsari River in Samarinda City, Samarinda Utara District, East Kalimantan Province, it has transformed from a natural water reservoir into a commercial and residential area. The consequence of this change is the frequent and regular flooding in the area.

Samarinda City, as a supporting city for the country's future capital, has experienced rapid population growth. This population explosion is a contributing factor that triggers the flood issues in Samarinda. The reduction of green areas resulting from land use changes affects the hydrological characteristics and the river's potential as a natural reservoir.

In general, the current condition of the Talangsari River can no longer accommodate the flood discharge, leading to widespread flooding in the Talangsari River Basin from upstream to downstream. This overflow is caused by high rainfall intensity, sedimentation accumulation, and the relatively small cross-sectional area of the Talangsari River, ranging from approximately 1 m to 3.5 m. Based on the topography of Samarinda City, about 30% of the areas or regions are situated below the river level, contributing to the high occurrence of flooding in Samarinda (Figure 1) [2]. The topographic map can be seen in Figure 3.

Considering the factors contributing to the flooding in the Talangsari River, river normalization is the best countermeasure to mitigate the flood issue due to the limited conditions. River normalization on the Talangsari

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River can be done by dredging and widening river dimensions to increase capacity.

In this study, the river length is 1.7 km, with a watershed area of 2.81 km^2. The Talangsari River mouth joins the Karangmumus River approximately 50 m from the PM Noor Bridge. The Talangsari River Basin exhibits a sub-dendritic flow pattern. This sub-dendritic flow pattern occurs when multiple tributaries or branches contribute, resembling branches of a tree merging into the main river. Along the Talangsari River are several main drainage points, including those from Jl. Talangsari, Jl. Mugirejo, the Citraland housing area, and the Alaya housing area. The Talangsari River Basin is a drainage system outlet for residential areas, commercial areas, or economic centres along Jl. DI Panjaitan, starting from Gunung Tangga/Talangsari to Simpang 3 Jl. DI Panjaitan in Samarinda. Figure 2 illustrates the Talangsari River Basin Map.

Figure 1 The location map of Talangsari River in Samarinda City, North Samarinda District, East Kalimantan. (Source: Google Earth)

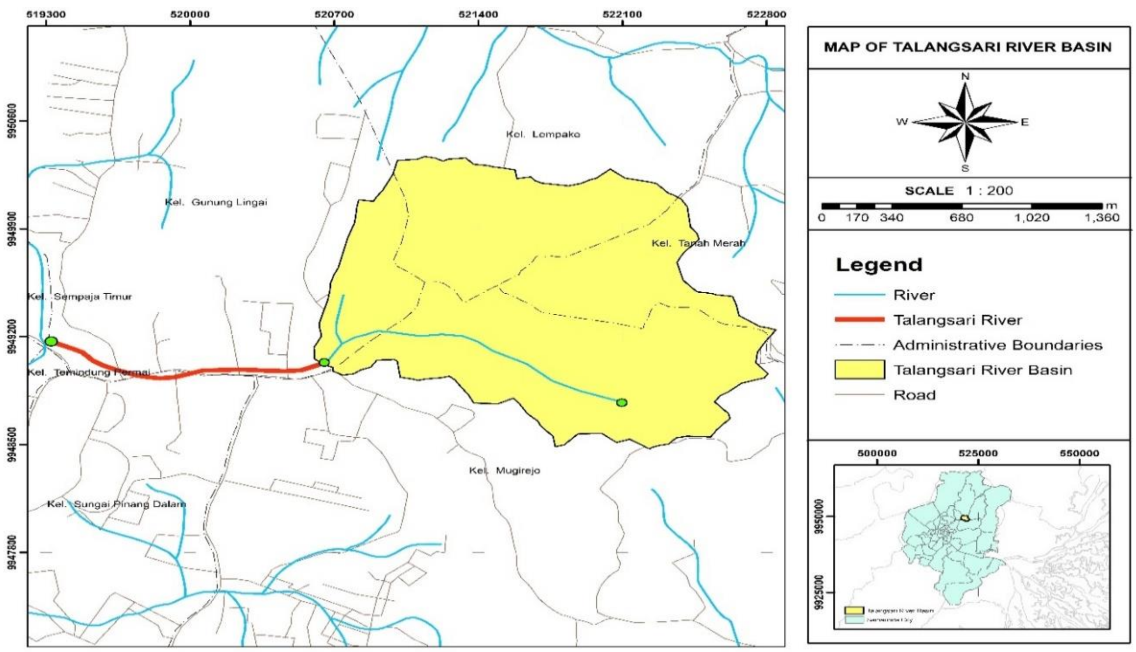

Figure 2 Map of the Talangsari Watershed in Samarinda City, North Samarinda District, East Kalimantan Province. (Map compiled based on DEMNAS data)

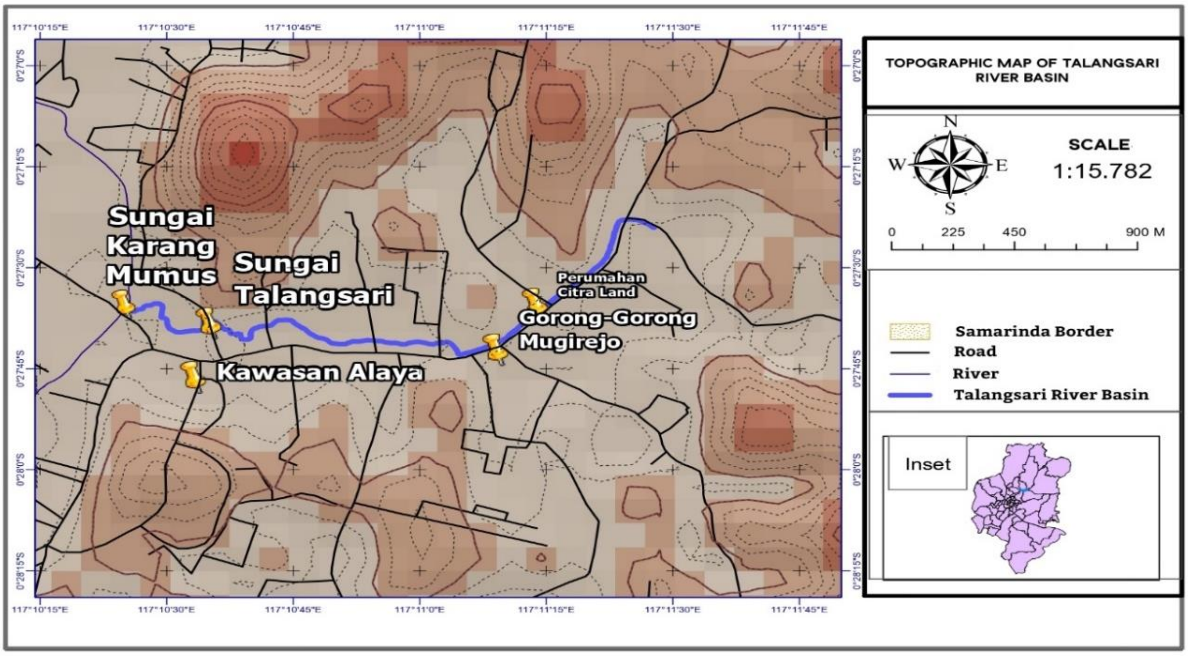

Figure 3 Topographic map of the Talangsari Watershed in Samarinda City, East Kalimantan Province.

2. Method

The research methodology includes hydrological analysis and hydraulic modelling using the HEC-RAS software. Data preparation is required in the hydrological analysis step, including rainfall data to calculate annual rainfall and design rainfall. The design rainfall is used to calculate the design flood discharge. Forty-three events of extreme rainfall data (greater than 50 mm/day) were collected to obtain the design rainfall by frequency analysis.

Among these events, the dominant rainfall duration is 11 hours, occurring in 7 rainfall events. The data for extreme rainfall events with depths exceeding 50 mm can be obtained using WRPlot or Excel to determine the frequency of each depth and select the most frequently occurring duration as the dominant duration. The distribution pattern calculation will utilise the data for rainfall events exceeding 50 mm.
Frequency analysis is conducted to determine the magnitude of design rainfall based on specific design criteria. Several distributions are commonly used in frequency analysis, including Normal, Log Normal, Gumbel, and Log Pearson III. Each distribution requires different data parameters, namely the coefficient of skewness (Cs) and coefficient of kurtosis (Ck), as specified in Table 1.

After performing the frequency analysis calculations, the next step involves conducting goodness-of-fit tests using the Chi-square and Smirnov-Kolmogorov tests. These goodness-of-fit tests determine whether the selected distribution type is appropriate for the given data.

Table 1 Statistic parameters requirement for probability distribution

No	Distribution	Requirement
1	Gumbel	$C_s = 1.14$ $C_k = 5.4$
2	Normal	$C_s \approx 0$ $C_k \approx 3$
3	Log Normal	$C_s = Cv^3 + 3C_v$ $C_k = Cv^8 + 6Cv^6 + 15Cv^4 + 16C_v^2 + 3$
4	Log Pearson III	Aside from the above values

Frequency analysis calculations can be performed using the following calculations:

$$x_T = \mu + K_T\sigma$$

where x_T is estimated value in a certain period, σ is standard deviation, K_T is frequency factor, and μ is the average value of occurance.

The frequency factor equation was developed by Ven Te Chow in 1951 [3] and is applicable to many probability distributions used in hydrological frequency analysis. For specific distributions, the K-T relationship can be established between the frequency factor and the corresponding return period. This analysis is performed using an Excel frequency analysis program [4].

After calculating the design rainfall using frequency analysis and conducting goodness-of-fit tests, the next step involves computation using the synthetic unit hydrograph method. The synthetic unit hydrograph methods used are Nakayasu and SCS. Synthetic unit hydrographs can be utilized when historical data is unavailable.

The hydraulic analysis is performed using the HEC-RAS software. The HEC-RAS program is used to calculate and simulate steady flow and unsteady flow conditions. The HEC-RAS simulation is conducted using a 1D flood modelling approach. The program also calculates water surface profiles along the river reach. Input data for the program include river cross-sections, longitudinal profiles, hydraulic parameters (Manning's roughness and slope), river structures, flow discharges, and water levels. The program outputs can consist of tables and graphs, including river schematics, cross-section plots, profiles, rating curves, and stage and flow hydrographs [5].

There are two approaches to mitigate the flood issue: structural and non-structural methods. Non-structural methods involve sediment removal to restore the river's optimum function, while structural methods involve the construction of embankments to prevent overflow. Structural methods are still the primary choice for flood management efforts [6]. The construction of embankments can only be carried out after non-structural methods have been implemented, but flooding persists [7]. As each phase of flood control works is completed, the capacity to handle flood discharges increases. Therefore, when the final phase of work is completed, the flood control system can function as planned. These efforts also significantly impact human society's economic, social, institutional, and environmental aspects [8].

3. Results

3.1 Hydrological Analysis

In this study, hydrological analysis is used to calculate the flow occurring in the Talangsari Watershed, specifically the design flood discharge.

In the rainfall-runoff analysis to estimate the design flood discharge, input of the design rainfall is required into a watershed system. The rainfall design can be in the form of rainfall depth at a specific point or a rainfall hyetograph, representing rainfall distribution as a function of time during extreme rainfall events [9]. In this study, the calculation of design rainfall uses maximum daily rainfall data for 20 years (2001-2020) obtained from the Temindung Rain Gauge Station (BMKG) [10]. The Temindung Rain Gauge Station is the closest station to the study area. The summary of annual maximum daily rainfall data can be seen in Table 2.

Table 2 Daily maximum rainfall data of Temindung Station

Year	Rainfall (mm)	Year	Rainfall (mm)
2001	61	2011	105
2002	66	2012	98
2003	76	2013	84
2004	118	2014	102
2005	108	2015	79
2006	132	2016	120
2007	94	2017	102
2008	132	2018	233
2009	74	2019	99
2010	86	2020	94

To determine the design rainfall, it is necessary to perform frequency analysis, which aims to establish the relationship between the magnitude of extreme events and their frequency using probability distributions, based on the annual maximum daily rainfall data as shown in Table 2. Commonly used probability distributions include Normal, Log Normal, Gumbel, and Log Pearson III. There are two methods to test whether the selected distribution is appropriate for the available data, namely the Chi-Square test and the Smirnov-Kolmogorov test [11].

Table 3 Chi-squared test results

Probability Distribution	Normal	Log Normal	Gumbel	Log Pearson III
χ^2 Calculated	6.5	3	4	1.5
χ^2_{Cr}	5.99	5.99	5.99	5.99
Information	rejected	Accepted	Accepted	Accepted

Table 4 Smirnov-Kolmogorov test results

Probability Distribution	Normal	Log Normal	Gumbel	Log Pearson III
Δ max	0.164	0.095	0.094	0.121
Δ criticism	0.29	0.29	0.29	0.29
Information	Accepted	Accepted	Accepted	Accepted

From Table 3 and Table 4, it is evident that the distributions yield different results. The Chi-Square test was conducted using the Normal distribution, while the Smirnov-Kolmogorov test was performed using the Log Pearson III distribution. Both distributions also differ in determining the magnitude of design rainfall. The design rainfall for a specific return period from both distributions can be observed in Table 5.

Table 5 Design rainfall with specific return period

Return Period (year)	P Pearson III Log Distribution (mm)	P Gumbel Distribution (mm)
1	71	63
2	94	97
5	123	130
10	146	151
25	180	178
50	210	198
100	243	218

The selected design flood return period should function effectively in terms of timing, structural considerations, and functionality [11]. In this study, the researcher opted for a 25-year return period, which would be used during the normalization simulation using the HEC-RAS program. As shown in Table 5, the design rainfall for a 25-year return period using the Log Pearson III distribution is 180 mm, while for the Gumbel distribution, it is 178 mm. Therefore, in this study, the design rainfall value to be used for further analysis is 180 mm, based on the Log Pearson III distribution. The Log Pearson III distribution was chosen because it yielded a higher value compared to the Gumbel distribution.

Extreme rainfall events at a rain gauge station can be calculated through observation using rainfall data from automatic stations. In this study, the rainfall data used consisted of extreme rainfall events over a 5-year period (2016-2020) obtained from JAXA, with the coordinates of the Temindung Rain Gauge Station, which is the nearest station to the research location [12] . The use of JAXA's extreme rainfall data can serve as a reference for extreme rainfall data as their global rainfall maps are highly accurate with high resolution [13]. The analysis of rainfall distribution patterns was conducted using data on dominant rainfall events with depths exceeding 50 mm, which are considered representative of the studied area. There were 42 events with rainfall depths exceeding 50 mm, and among them, 7 events had a dominant duration of 11 hours. To determine the dominant duration, software like WRPlot or Excel can be utilized to identify the most frequently occurring depth values for each hour, and the duration with the highest frequency is selected as the dominant duration. After conducting the rainfall distribution analysis, the rainfall intensity for a 2-year return period was determined to be 94 mm, while for a 25-year return period, it was found to be 180 mm. The analysis data for rainfall exceeding 50 mm is summarized in Table 6, and the rainfall distribution pattern is presented in Table 7, with graphical representations shown in Figure 4 and Figure 5.

Table 6 Rainfall distribution patterns

time	Intensity P_{25} 180 mm	Intensity P_2 94 mm
0	0	0
1	5.52	2.88
2	13.31	6.95
3	19.79	10.34
4	24.02	12.54
5	27.92	14.58
6	25.82	13.48
7	22.87	11.94
8	19.19	10.02
9	11.74	6.13
10	6.76	3.53
11	3.06	1.60
∑	180	94

Table 7 Summary of rainfall distribution pattern > 50 mm

t	Σ t (%)	ΣP (%)					% Average ΣP	%P
		1	2	(...)	41	42		
0	0	0	0		0	0	0	0
1	9.1	0.1	0.0		1.6	3.4	3.1	3.1
2	18.2	3.7	0.5		5.4	15.7	10.5	7.4
3	27.3	4.9	2.9		9.9	31.3	21.5	11.0
4	36.4	6.5	7.6		10.8	46.9	34.8	13.3
5	45.5	9.6	18.3		15.4	61.3	50.3	15.5
6	54.5	15.5	36.9	(...)	27.7	73.9	64.7	14.3
7	63.6	39.2	57.7		47.6	82.8	77.4	12.7
8	72.7	70.3	76.3		73.2	88.2	88.0	10.7
9	81.8	88.4	91.5		90.5	91.8	94.5	6.5
10	90.9	95.9	99.2		98.1	95.5	98.3	3.8
11	100	100	100		100	100	100	1.7

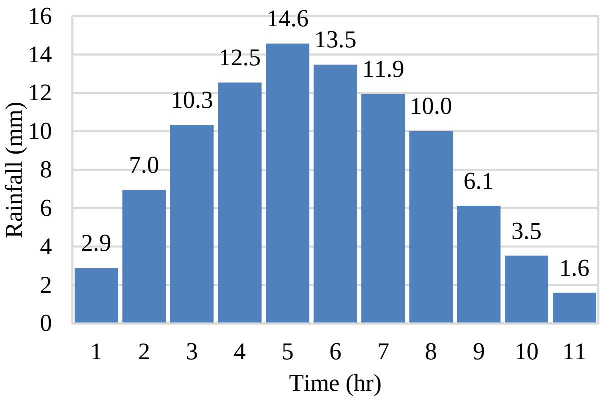

Figure 3 The 2-Year rainfall distribution pattern

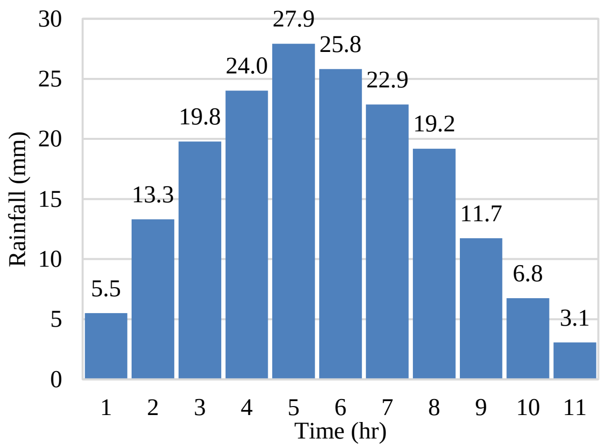

Figure 4 The 25-Year rainfall distribution pattern

The parameters used in the unit hydrograph SCS and Nakayasu methods are the watershed area, river length, and river slope in the watershed. Based on the obtained data, the Talangsari watershed has an area of 2.8 km², a river length of 1.7 km, and a river slope of 0.044. With these parameters, the analysis results for the unit hydrograph SCS method are presented in Table 8 and Figure 6, while the results for the Nakayasu method are shown in Table 9 and Figure 7.

Table 8 Nakayasu unit hydrograph of Talangsari Watershed

Data		
A	1.7	Miles
L	2.81	km^2
Re	1	Mm
Analysis		
t_g	0.499	hour
t_r	0.374	hour
T_p	0.798	hour
α	2	
$T_{0.3}$	0.997	hour
Qp	0.631	m^3/s
$T_p+H_{0.3}$	1.795	hour
$T_p+H\ 0.3+1.5T_{0.3}$	3.291	hour

Table 9 SCS unit hydrograph of DAS Talangsari

Data		
Watershed Area (A)	2.81	km^2
Main River Length (L)	1.7	Miles
Average River Slope (S)	0.044	
Analysis		
Concentration Time (Tc)	0.326	hour
Time Lag (tp)	0.196	hour
Duration of Effective Rain (tr)	0.043	hour
Peak Time (Tp)	0.218	hour
Base Time (Tb)	0.581	hour
Peak Discharge (Qp)	2.687	m^3/s

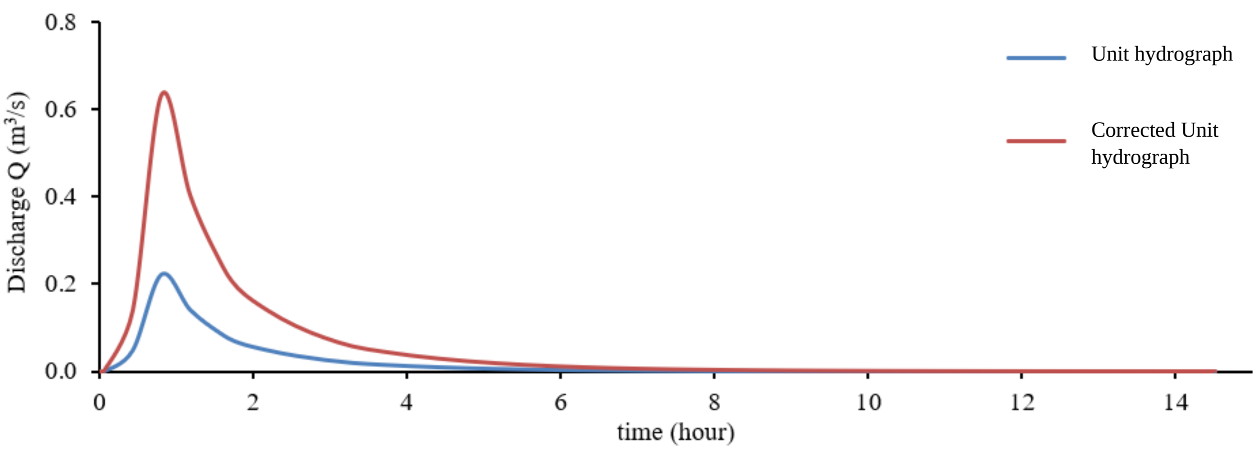

Figure 5 Nakayasu's unit hydrograph Graph

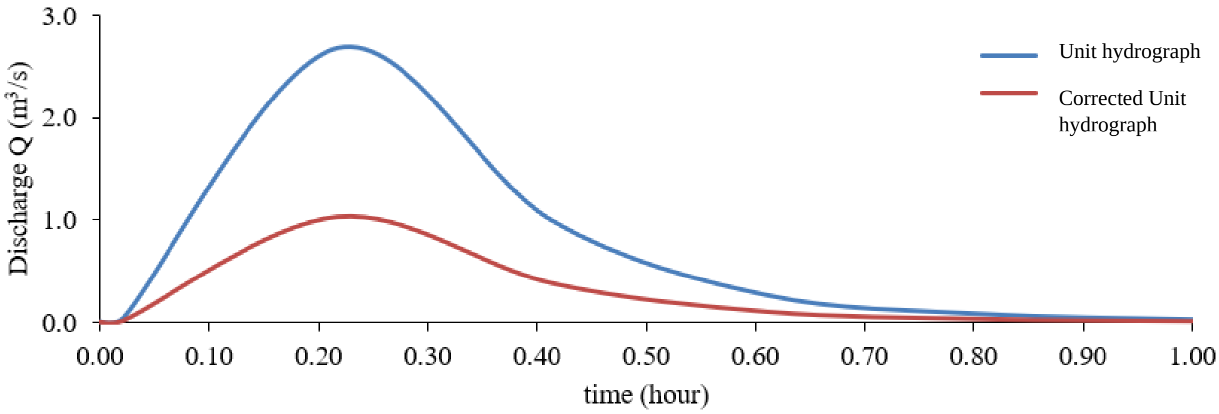

Figure 6 SCS unit hydrograph

Effective rainfall is a portion of excess rainfall that results in direct runoff. The magnitude of effective rainfall is influenced by land conditions, watershed characteristics, and the amount of rainfall occurring in the river basin. To calculate losses using the SCS-CN method, several calculation parameters are required, including the Curve Number (CN) derived from land use data and soil classification. In this study, soil type C is used, which represents soil with moderately high runoff potential and slow infiltration rate when the soil is fully saturated.

From the loss analysis, the following values were obtained: CN II = 81.57, initial abstraction (Ia) = 4.66, and retention parameter (S) = 23.28 mm. These loss parameter calculations are used to calculate the effective rainfall (P_{eff}) at the study location. The calculation of effective rainfall is based on the results of previous analyses, including the distribution pattern of rainfall, design rainfall calculation, and loss calculation. The results of the effective rainfall calculation can be seen in Table 10 and Table 11.

The flood hydrograph calculation is used to determine the design flood discharge. The calculation is performed by multiplying the unit hydrograph with the flood hydrograph. After conducting the flood hydrograph calculation, peak discharges for the 2-year and 25-year return periods are obtained, which will be used for simulation in the HEC-RAS program. The summary of peak discharges can be seen in Table 12.

3.2 Hydraulic Modeling with HEC-RAS

The hydraulic modeling simulation aims to analyze the channel's capacity to accommodate the given flow discharge. In this study, the simulation is conducted with two scenarios: the existing condition simulation and the simulation with normalization.

Table 10 Two-years effective rainfall

t	P_2 94 mm	∑ P	∑ P_{eff}	P_{eff}
1	2.8	2.8	1.5	1.5
2	6.9	9.8	0.04	-1.4
3	10.3	20.1	1.1	1.0
4	12.5	32.7	5.7	4.5
5	14.5	47.2	13.7	8.0
6	13.4	60.7	22.7	9.0
7	11.9	72.7	31.6	8.8
8	10.0	82.7	39.4	7.8
9	6.1	88.8	44.4	4.9
10	3.5	92.4	47.3	2.9
11	1.5	94.0	48.6	1.3

Table 11 Twenty five-years effective rainfall

t	P_{25} 180	∑ P	∑ P_{eff}	P_{eff}
1	5.5	5.5	0.7	0.7
2	13.3	18.8	0.8	0.1
3	19.7	38.6	8.7	7.9
4	24.0	62.6	24.1	15.4
5	27.9	90.5	45.8	21.7
6	25.8	116.3	67.8	22.0
7	22.8	139.2	88.2	20.3
8	19.1	158.4	105.7	17.5
9	11.7	170.1	116.6	10.8
10	6.7	176.9	122.7	6.2
11	3.0	180.0	125.7	2.9

Table 12 Peak discharge

Peak Discharge (m^3/s)			
Unit Hydrograph Nakayasu		Unit Hydrograph SCS	
Q_2	Q_{25}	Q_2	Q_{25}
6.39	13.39	15.09	34.44

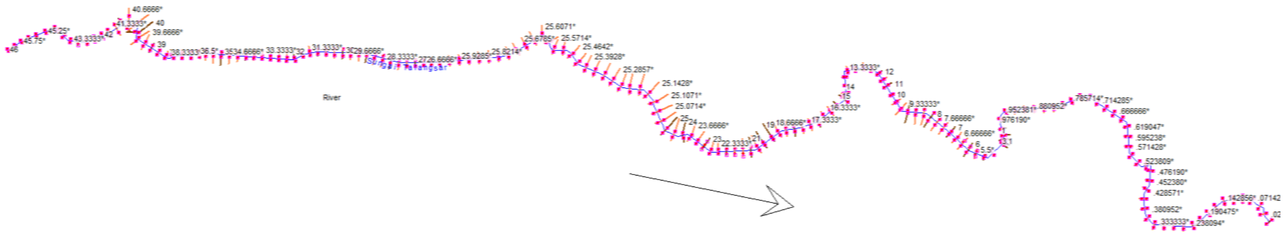

Figure 7 Talangsari river geometry input

The existing flow simulation aims to determine the capacity of the existing channel cross-section to receive and convey the flood discharge, allowing the identification of overflowing sections. The first step in the flow simulation using HEC-RAS is inputting the geometry data. The cross-sectional and longitudinal profiles of the river are created and inputted based on field measurement data, while the Manning's roughness coefficient (n value) is adjusted based on direct field investigations [14]. A higher Manning's roughness coefficient indicates a rougher surface. The Manning's n values for each section of the channel are shown in Table 13, and the geometry of the Talangsari Watershed can be seen in Figure 8.

Table 13 The Manning's roughness coefficient (n value) of the Talangsari River

Point	Value n manning		
	LOB	Channel	ROB
Sta. 0+000 - sta. 0+060	0.018	0.018	0.018
Sta. 0+060 - sta. 0+082	0.022	0.022	0.022
Sta. 0+082 - sta. 0+257	0.03	0.03	0.03
Sta. 0+257 - sta. 0+282	0.027	0.027	0.027
Sta. 0+282 - sta. 0+332	0.025	0.025	0.025
Sta. 0+332 - sta. 0+357	0.027	0.027	0.027
Sta. 0+357 - sta. 1+700	0.03	0.03	0.03

In this study, the flow modeling using HEC-RAS is simulated with unsteady flow conditions. Unsteady flow simulation is chosen to understand the changes in flow over time, aiming to depict flood events closely resembling real occurrences at the research site. Unsteady flow simulation requires two boundary conditions: upstream boundary conditions and downstream boundary conditions. For upstream boundary conditions, the simulation utilizes a hydrograph with a 2-year return period discharge. It is assumed that the 2-year return period discharge can represent the existing flood events. As for downstream boundary conditions, tidal data is used for the modeling, specifically the tidal stage hydrograph at the downstream of the Talangsari River. The tidal data is incorporated into HEC-RAS as a stage hydrograph. The tidal stage utilized corresponds to the tidal range between the confluence of the downstream of the Talangsari River and the Karang Mumus River, with a magnitude of +5.2 meters.

Based on the available data, the simulation of the existing conditions is conducted using the HEC-RAS software. In the simulation of the existing conditions, the river is subjected to a discharge of 1.28 m^3/s. Additionally, the Froude number at the maximum simulated discharge indicates a value below 1 ($fr < 1$), indicating that the flow type in the Talangsari River is subcritical. The longitudinal profile (long section) of the river in Figure 12 demonstrates the fluctuating flow, varying channel shapes that result in flow constriction, and potentially influence the upstream flow. The cross-sectional profiles of the Talangsari River can be observed in Figures 9 to Figure 11.

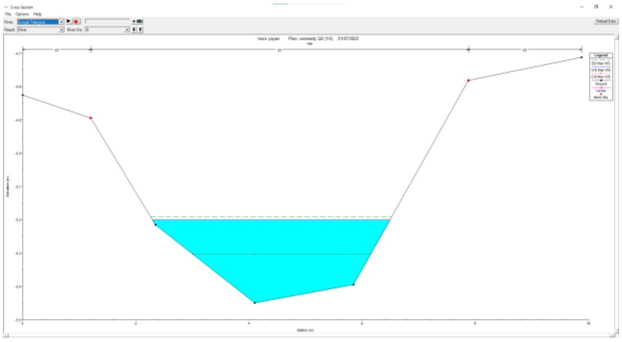

Figure 8 Cross-sectional profile of the Talangsari River at Sta. 0+000

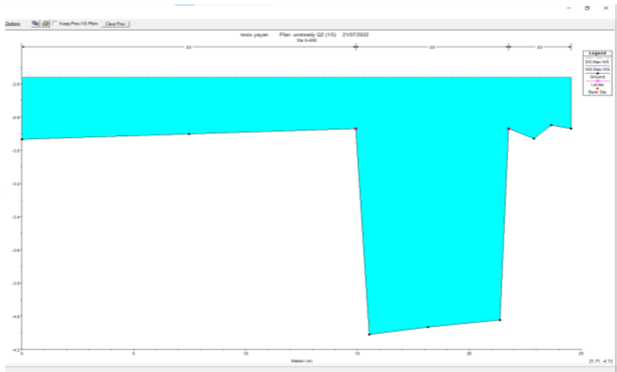

Figure 9 Cross-sectional profile of the Talangsari River at Sta. 0+762

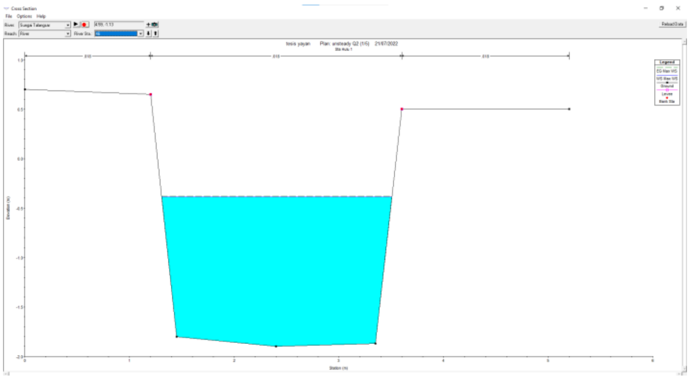

Figure 10 Cross-sectional profile of the Talangsari River at Sta. 1+700

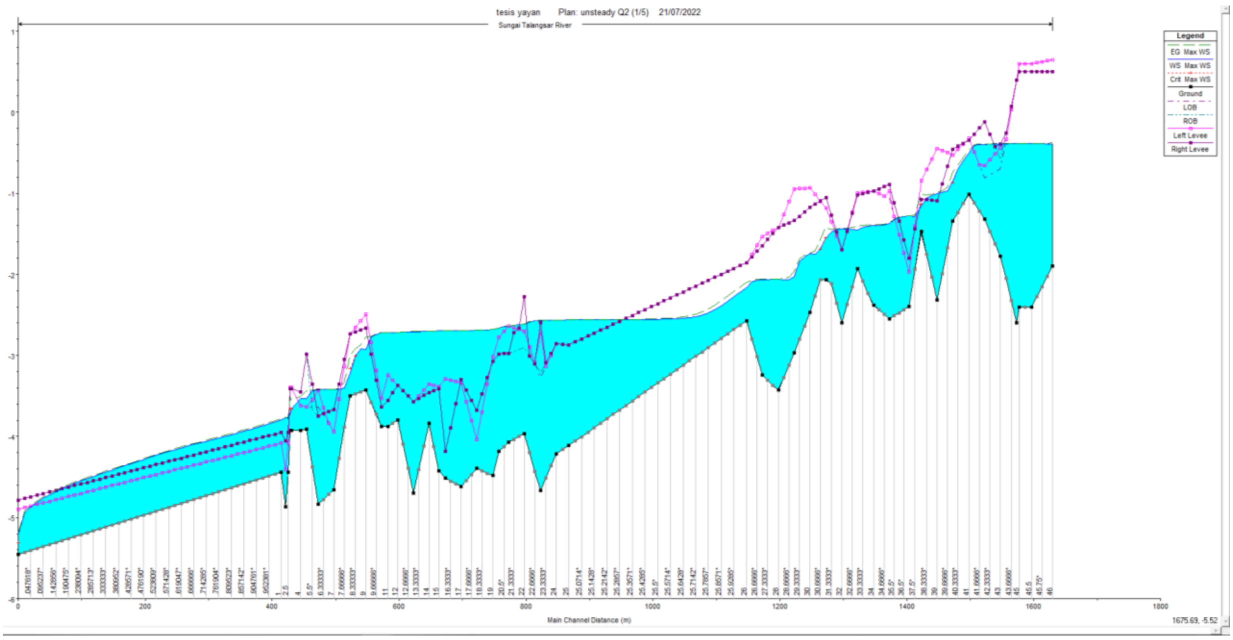

Figure 11 Longitudinal profile of the Talangsari River during the existing simulation.

Based on the simulation of the existing conditions, it is evident that the entire Talangsari River experiences flooding. Therefore, it is necessary to conduct river normalization along the entire stretch of the Talangsari River due to significant morphological changes, such as channel narrowing and bed sedimentation, as depicted in Figure 12. The normalization simulation will cover the reach from station 0+000 to station 1+700, utilizing a 25-year return period discharge of 13.38 m^3/s. The proposed channel shape for the normalization of the Talangsari River is rectangular, with a bottom width of 7 m, channel depth of 2.5 m, and a guard height of 1 m. The normalization will utilize concrete as the construction material, and the Manning's roughness coefficient (n value) used for the simulation is 0.013. The cross-sectional profiles and longitudinal profile of the Talangsari River for the normalization can be observed in Figure 13 to Figure 16.

After the normalization process with a 25-year return period discharge, Sungai Talangsari no longer experiences flooding. This indicates that the performed normalization was successful in addressing the flood issue in the river. Normalization of the river is crucial in increasing the channel capacity and reducing flood risk in the area. By optimizing the dimensions of the channel, river structures, and minimizing flow obstructions, normalization can help control water flow and mitigate the risk of flooding.

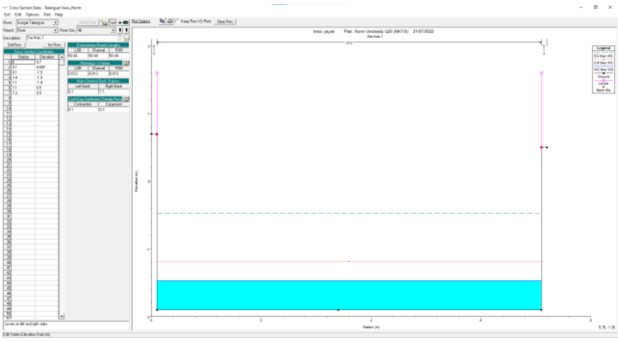

Figure 12 Cross-sectional profile of the Talangsari River at Sta. 0+000 after normalization

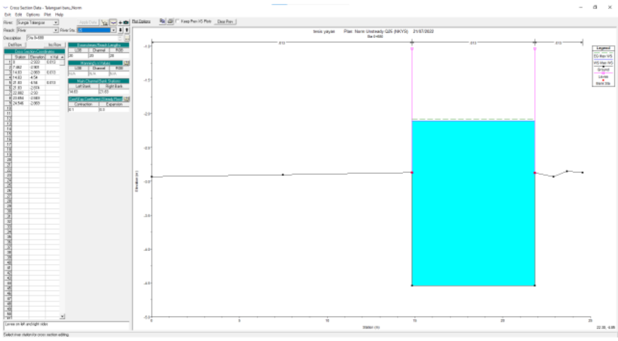

Figure 13 Cross-sectional profile of the Talangsari River at Sta. 0+762 after normalization

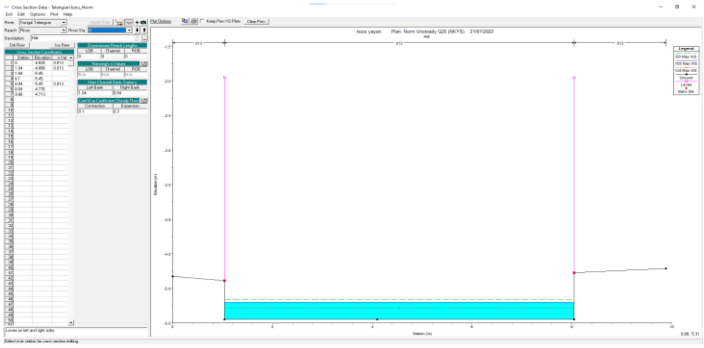

Figure 14 Cross-sectional profile of the Talangsari River at Sta. 1+700 after normalization

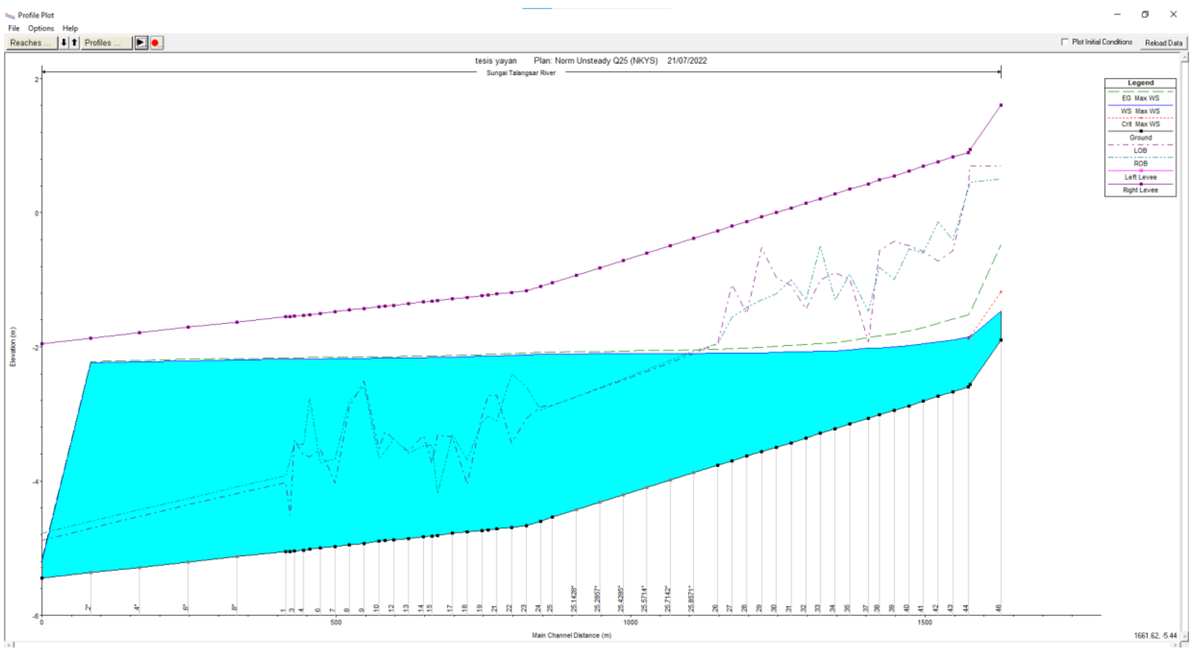

Figure 15 Longitudinal profile of the Talangsari River after normalization.

4. Conclusion

Based on the research findings, the peak discharge for the existing condition (Q_2) was determined to be 6.39 m^3/s, while the peak discharge for the normalization scenario (Q_{25}) was found to be 13.38 m^3/s. However, during the simulation using these discharges, it was observed that Sungai Talangsari could not convey and accommodate the increased flow, highlighting the need for further simulation to ensure Sungai Talangsari's capacity to handle larger discharges.

A comprehensive normalization approach was implemented along the entire stretch of the river, employing a rectangular channel shape with a bottom width of 7 m, a channel depth of 2.5 m, and a guard height of 1 m. Concrete materials were selected to normalize Sungai Talangsari, with a Manning's roughness coefficient (n value) of 0.013. After conducting the normalization modeling, Sungai Talangsari effectively handled a discharge of 13.38 m^3/s (Q_{25}).

Additionally, proactive measures can be taken by the community to prevent flooding, such as providing training to enhance preparedness and resilience during emergency situations. Active participation and engagement of the community are crucial aspects in effectively mitigating the impacts of floods in civil engineering projects [15].

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