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Analysis of Urban Drainage System Based on Ponorogo City Spatial Planning

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Abstract

Urban flooding in Ponorogo Regency remains a persistent infrastructure challenge, primarily due to inadequate drainage systems in key urban areas such as Jl. Pramuka, Jl. Jaksa Agung Suprapto, Jl. Bhayangkara, and Jl. Kbp Duriyat. This study aims to analyze rainfall patterns, determine flood discharge levels, and evaluate the capacity of existing drainage infrastructure to propose effective improvements. The research employed a 20-year rainfall dataset from the Ponorogo Rainfall Station, analyzed using the Log Pearson Type III method, which was validated through Normality Tests based on the Smirnov-Kolmogorov and Chi-Squared criteria. The 10-year design rainfall was determined to be 111.68 mm. Using rational calculations, the 10-year flood discharge ranged from 0.520 m^3/s (Jl. Bhayangkara Kanan) to 0.949 m^3/s (Jl. Pramuka Kiri). Findings reveal that the existing drainage channels are insufficient to accommodate the calculated discharge, requiring upgrades. Recommended improvements include U-Ditch 100×100 cm for Jl. Pramuka and Jl. Suprapto, and U-Ditch 80×80 cm for Jl. Bhayangkara and Jl. Kbp Duriyat. These enhancements are expected to significantly reduce urban flood risks and improve drainage performance.

Keywords : *Drainage, Urban, Flooding, Channel.*

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Introduction

A pivotal element of urban development pertains to the provision of infrastructure. Urban infrastructure is defined by the presence of clean water systems, sanitation systems, and drainage systems (Kurniawan et al., 2023). In order to ensure the ongoing sustainability of water resource utilization and to optimize the utilization of facilities and resources, it is necessary to implement the following measures (Kurdi et al., 2020). The expansion of infrastructure within *Ponorogo District*, coupled with the region's annual population growth, has resulted in an escalation of activities and requirements. The conversion of rainwater absorption areas into residential zones has been a prominent land-use change, resulting in diminished rainwater absorption and increased flood susceptibility. This alteration has diminished the land's capacity for natural water absorption (Xu et al., 2015).

The infrastructure problem in the Ponorogo area is characterized by flooding in the city of Ponorogo, which is attributable to the urban drainage system. An imbalance between inflow and outflow, with inflow exceeding outflow, has been demonstrated to cause flooding (Hasanah et al., 2021). This is especially prevalent when water channels and absorption areas are not functioning properly (Tanjung & Setiawan, 2020). Drainage facilities are engineered to address the necessity for water disposal (Kurniawan et al., 2020). Flooding issues have been observed in the city of Ponorogo, specifically on the following roads: Jl. Pramuka, Jl. Jaksa Agung Suprapto, Jl. Bhayangkara, and Jl. Kbp Duriyat (Wahyu & Purnama, 2021). In view of the issues identified, it is imperative that a thorough analysis and evaluation of the urban drainage system be conducted, with the city's spatial planning master plan serving as the overarching framework (Prasetyo & Hidayati, 2020). Urban spatial planning is of pivotal significance in the realm of flood control (Aziz & Sari, 2021). Comprehensive, environmentally conscious planning that considers the hydrological characteristics of the area can significantly reduce the risk and impact of flooding in the city of Ponorogo (Rahayu & Kuswandi, 2020). Consequently, a rigorous analysis of the design flood discharge calculations is imperative, with a view to assessing their congruence with the stipulated storage capacity as delineated within the Ponorogo City Spatial Planning (Mahmudi et al., 2020). This kind of assessment ensures the efficacy of flood management strategies (Sari & Suryani, 2021).

A pivotal element of sustainable urban development is the availability of reliable infrastructure, including clean water, sanitation, and drainage systems (Kurniawan et al., 2023). Inadequate drainage infrastructure can significantly increase the risk of urban flooding, particularly in areas undergoing rapid population growth and land-use changes. In *Ponorogo District*, the annual population increase and the conversion of rainwater absorption areas into residential and commercial zones have drastically reduced natural infiltration capacity, thereby heightening flood vulnerability. This issue highlights the urgent need for improved urban drainage systems and water management strategies (Kurdi et al., 2020).

Previous studies, such as Hasanah et al. (2021), have shown that flooding in urban areas is frequently caused by an imbalance between surface runoff and drainage outflow, often exacerbated by poorly functioning water channels and reduced absorption areas. However, few studies have systematically analyzed the hydrological conditions and design flood discharge capacities specific to *Ponorogo's* urban drainage system. This gap necessitates a localized, data-driven evaluation to inform effective flood mitigation measures.

This research aims to assess the adequacy of current drainage infrastructure, propose optimized designs using *U-Ditch* systems, and ensure alignment with *Ponorogo's* spatial planning master plan. The benefits include actionable recommendations for infrastructure improvement, flood risk reduction, and environmentally sustainable urban planning.

Research Method

The research commenced with a literature study, the objective of which was to collect and review relevant materials. The literature study was conducted to facilitate an understanding of the theoretical underpinnings and conceptual framework of the research. The materials included in the literature study comprised books and scientific journals. Subsequently, both primary and secondary data were collected. The research data was then

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analyzed in order to determine the selected rainfall data. This was achieved by employing frequency distribution analysis as the basis for conducting the hydrological analysis. The *rational method* was utilized in the execution of hydrological discharge calculations. Next, a hydraulic analysis was conducted to evaluate the storage capacity of the existing channel. The subsequent stage involved enhancing the cross-section of the existing channel in accordance with the planned hydrological discharge. Afterwards, the processing of *geographical information systems* was conducted to generate a drainage map database, utilizing *ArcGIS* software. The water surface profile of the drainage channel, both before and after improvement, was determined using *HEC-RAS* software.

Results and Discussion

Distribution Type Requirements

The distribution calculations were performed on the basis of rainfall data from the Ponorogo Station for the period 2005-2024. Distribution calculations are a means of quantifying variability or frequency, thereby establishing the extent of variation or frequency. As illustrated in Table 1, the results of the distribution type requirement calculations are presented:

Table 1. Distribution Type Requirements

No	Distribution Type	Requirements	Calculation Results	Conclusion
1	Gumbel	$Cs \approx 1{,}1306$	2,825	Does not meet
		$Ck \approx 5{,}40$	12,773	
2	Log Normal	$Cs = Cv^3+3Cv$	1,159	Does not meet
		$Ck = Cv^8 + 6Cv^6 + 15Cv^4 + 16Cv^2 + 3$	5,481	
3	Normal	$Cs \approx 0$	2,825	Does not meet
		$Ck \approx 3$	12,773	
4	Log Pearson Type III	As well as the above values $Cs \neq 0$	2,825	Meets

Source: Analysis Results, 2025

The skewness coefficient (C_s) is utilized to quantify the extent of asymmetry in the distribution shape. Conversely, the kurtosis coefficient (C_k) is utilized to ascertain the peak value of the data distribution. As demonstrated in Table 1, it can be deduced that the distribution calculation category in question employs the Log Pearson Type III method.

Normality Test

Chi-Square Test

The Chi-Square test was employed to ascertain the presence of data deviation in the sample. Utilizing a degree of freedom of 1,67 and a confidence level of 1%, the $C2_{cr}$ value was determined to be 7,682. As illustrated in Table 2, the results of the Chi-Square Test calculation are as follows:

Table 2. Chi Square Test Calculation

Probability Distribution	c2 calculated	c2 $_{cr}$	Requirements	Description
Gumbel	11,800	7,682	$c2 > c2_{cr}$	Does not meet
Normal	13,000	7,682	$c2 > c2_{cr}$	Does not meet
Log Normal	6,400	7,682	$c2 < c2_{cr}$	Meets
Log Pearson Type III	6,400	7,682	$c2 < c2_{cr}$	Meets

Source: Analysis Results, 2025

As illustrated in Table 2, the distribution types obtained are Log Normal and Log Pearson Type III. This is due to the fact that the calculated c2 values for both distributions meet the requirement ($c2<c2_{cr}$) with values of 6,400 < 7,682. Conversely, for the Gumbel and Normal distributions, the calculated c2 value exceeds the $c2_{cr}$ requirement of 7,682.

Kolmogorov-Smirnov Test

The Kolmogorov-Smirnov test is utilized to ascertain whether the hypothesis being tested is normally distributed or equivalent to the theoretical distribution. Utilizing a data set (n) of 20 and a confidence level of 1%, the D_{pcr} value is determined to be 0,352. The results of the Kolmogorov-Smirnov test are presented in Table 3 below:

Table 3. Kolmogorov-Smirnov Test Calculation

Probability Distribution	D_{pmax}	D_{pcr}	Requirements	Description
Gumbel	0,716	0,352	$D_{Pmax} > D_{Pcr}$	Does not meet
Normal	0,752	0,352	$D_{Pmax} > D_{Pcr}$	Does not meet
Log Normal	0,823	0,352	$D_{Pmax} > D_{Pcr}$	Does not meet
Log Pearson Type III	0,064	0,352	$D_{Pmax} < D_{Pcr}$	Meets

Source: Analysis Results, 2025

As illustrated in Table 3, the distribution of rainfall calculations is derived using the Kolmogorov-Smirnov Test, employing the Log Pearson Type III method. This is due to the fact that the value of $D_{Pmax} < D_{Pcr}$, specifically 0,064 < 0,352. However, it was observed that the results obtained for probability distributions employing alternative methods were not satisfactory, as the D_{Pmax} value exceeded the D_{Pcr} value that had been established. Table 4 provides a summary of the frequency calculations for the Chi-Square Test and the Smirnov-Kolmogorov Test:

Table 4. Recapitulation of Normality Tests

No	Probability Distribution	Chi-Square Test	Kolmogorov-Smirnov Test
1	Gumbel	Does not meet	Does not meet
2	Normal	Does not meet	Does not meet
3	Log Normal	Meets	Does not meet
4	Log Pearson Type III	Meets	Meets

Source: Analysis Results, 2025

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As illustrated in Table 5, the results of the rainfall calculations performed using various distribution methods are presented, with these calculations being based on observations from the Ponorogo Rainfall Station:

Table 5. Rainfall Calculation Results

Repeat Period	Method				Rainfall Selected (mm)
	Gumbel	Normal	Log Normal	Log Pearson Type III	
2	105,384	111,475	106,496	105,487	105,487
5	149,326	146,087	135,584	109,057	109,057
10	178,416	164,217	153,867	111,681	111,681
20	205,958	179,051	170,644	114,559	114,559
25	215,182	183,584	176,127	115,144	115,144
50	242,453	195,946	191,991	117,787	117,787
100	269,523	207,483	208,084	120,460	120,460

Source: Analysis Results, 2025

The Log Pearson Type III method was selected for the calculation of rainfall at the Ponorogo Rainfall Station. Following a thorough frequency analysis and rigorous statistical testing using the Chi-square Test and Kolmogorov-Smirnov Test, Ponorogo City is classified as a medium city. In accordance with Appendix 1 of Ministry of Public Works Regulation No. 12/PRT/M/2014 on the Implementation of Urban Drainage Systems for Medium-Sized Cities with a catchment area > 500 Ha, the rainfall return period can be planned to be 5–10 years. In this study, a return period of 10 years was planned with a design discharge of 111,681 mm.

Hydrological Analysis

The design flood discharge is calculated based on the rational formula Q=0,278 C×I×A (km^2). The flow coefficient (C) value is 0,600 (surrounding area). The results of this flood discharge calculation are used to design the drainage system. As illustrated in Table 6, the results of the design discharge calculation are as follows:

Table 6. Rational Discharge Calculation

No	Channel Name	Time (t_c)	A (km^2)	I (mm/hour)	Q_{10} (m^3/ second)
1	Jl. Pramuka Kanan	0,881	0,125	42,137	0,879
2	Jl. Pramuka Kiri	0,881	0,135	42,137	0,949
3	Jl. Jaksa Agung Suprapto Kanan	0,393	0,055	72,155	0,662
4	Jl. Jaksa Agung Suprapto Kiri	0,393	0,060	72,155	0,722
5	Jl. Bhayangkara Kanan	0,490	0,050	62,280	0,520
6	Jl. Bhayangkara Kiri	0,490	0,058	62,280	0,654
7	Jl. Kbp Duriyat Kanan	0,349	0,044	78,153	0,574
8	Jl. Kbp Duriyat Kiri	0,349	0,050	78,153	0,652

Source: Analysis Results, 2025

As demonstrated in Table 6, the highest Q_{10} discharge value was identified on Jl. Pramuka Kiri at 0,949 m^3/second with an intensity of 42,137 mm/hour. Meanwhile, the lowest Q_{10} discharge value was found on Jl. Bhayangkara Kanan at 0,520 m^3/second with an intensity of 62,280 mm/hour.

Hydraulic Analysis

The capacity of existing channels is determined by hydraulic analysis. Furthermore, hydraulic analysis is utilized to ascertain the volume of water retained by existing channels, with the objective of preventing overflow. As illustrated in Table 7, the results of the current channel dimension calculations are presented:

Table 7. Existing Channel Dimensions

No	Name	Type	Wet Cross-Section Area				P (m^2)
			b (m)	h (m)	m	A (m^2)	
1	Jl. Pramuka Kanan	Trapezium	0,600	0,800	0,150	0,900	2,220
2	Jl. Pramuka Kiri	Trapezium	0,600	0,800	0,150	0,900	2,220
3	Jl. Jaksa Agung Suprapto Kanan	Quadrilateral	0,600	0,400		0,240	1,400
4	Jl. Jaksa Agung Suprapto Kiri	Quadrilateral	0,600	0,400		0,240	1,400
5	Jl. Bhayangkara Kanan	Quadrilateral	0,700	0,500		0,350	1,700
6	Jl. Bhayangkara Kiri	Quadrilateral	0,700	0,500		0,350	1,700
7	Jl. Kbp Duriyat Kanan	Quadrilateral	0,600	0,500		0,300	1,600
8	Jl. Kbp Duriyat Kiri	Quadrilateral	0,600	0,500		0,300	1,600

Source: Analysis Results, 2025

The calculation of the wet cross-sectional area and hydraulic radius of Jl. Pramuka Kanan (Trapezium) is illustrated in Table 7 as follows:

Area (A) $= \frac{(b+h\times m)}{h}$

$= \frac{(0{,}600+0{,}800\times 0{,}150)}{0{,}800}$

$= 0{,}900\ m^2$

Wet Cross-Section Perimeter (P) $= b+2h\sqrt{1+m^2}$

$= 0{,}600+2.0{,}800\sqrt{1+0{,}150^2}$

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= 2,220 m^2

As illustrated in Table 7, hydraulic calculations were subsequently conducted on the existing channel to ascertain its storage capacity. The value of n (*Manning Coefficient*) was determined to be 0,030 for concrete channels with a rough surface and uneven joints. The results of the hydraulic calculations for the channel are presented in Table 8 below:

Table 8. Hydraulic Calculations for Existing Channels

No	Name	R	n	s	V (m/second)	$Q_{tampung}$ (m^3/ second)
1	Jl. Pramuka Kanan	0,400	0,030	0,0027	0,945	0,850
2	Jl. Pramuka Kiri	0,400	0,030	0,0027	0,945	0,850
3	Jl. Jaksa Agung Suprapto Kanan	0,170	0,030	0,0054	0,756	0,181
4	Jl. Jaksa Agung Suprapto Kiri	0,170	0,030	0,0054	0,756	0,181
5	Jl. Bhayangkara Kanan	0,210	0,030	0,0028	0,616	0,216
6	Jl. Bhayangkara Kiri	0,210	0,030	0,0028	0,616	0,216
7	Jl. Kbp Duriyat Kanan	0,190	0,030	0,0060	0,845	0,253
8	Jl. Kbp Duriyat Kiri	0,190	0,030	0,0060	0,845	0,253

Source: Analysis Results, 2025

The following example illustrates the hydraulic calculation for the existing channel on Jl. Pramuka Kanan, as outlined in Table 8:

Flow Rate (V) $= \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$

$= \frac{1}{0,030} 0,400^{\frac{2}{3}} 0,0027^{\frac{1}{2}}$

= 0,945 m/second

Debit ($Q_{tampung}$) = A × V

= 0,900 × 0,945

= 0,850 m^3/ second

As illustrated in Table 8 above, subsequent calculations were conducted for the dirty water and the surrounding channel discharge (Q_{sal}) of the drainage channel under investigation. As illustrated in Table 9, the results of the total discharge calculations are presented:

Table 9. Hydrological and Hydraulic Analysis Calculations

No	Channel Name	Q_{hujan} (m^3/detik)	Q_{sal} (m^3/detik)	Q_{total} (m^3/ detik)	$Q_{tampung}$ (m^3/ detik)	Description
1	Jl. Pramuka Kanan	0,879	0,0687	0,947	0,850	Does not meet
2	Jl. Pramuka Kiri	0,949	0,0003	0,949	0,850	Does not meet

No	Channel Name	Q_{hujan} (m³/detik)	Q_{sal} (m³/detik)	Q_{total} (m³/ detik)	$Q_{tampung}$ (m³/ detik)	Description
3	Jl. Jaksa Agung Suprapto Kanan	0,662	0,0522	0,714	0,181	Does not meet
4	Jl. Jaksa Agung Suprapto Kiri	0,722	0,0001	0,722	0,181	Does not meet
5	Jl. Bhayangkara Kanan	0,519	0,0004	0,520	0,216	Does not meet
6	Jl. Bhayangkara Kiri	0,603	0,0517	0,654	0,216	Does not meet
7	Jl. Kbp Duriyat Kanan	0,574	0,0003	0,574	0,253	Does not meet
8	Jl. Kbp Duriyat Kiri	0,652	0,0003	0,652	0,253	Does not meet

Source: Analysis Results, 2025

The following example illustrates the hydraulic calculation for the existing channel on Jl. Pramuka Kanan, as outlined in Table 9:

Q_{total} = $Q_{10} + Q_{sal}$
= 0,879 + 0,947
= 0,947 m³/second

It is evident that, given the observation that $Q_{total} > Q_{tampung}$, the current channel is incapable of accommodating the total discharge. Consequently, it is necessary to initiate channel repairs. As illustrated in Table 10, the outcomes of the drainage channel repairs are evident:

Table 10. Channel Cross-Section Improvements

No	Nama Saluran	Q_{total} (m³/detik)	Q_{uditch} (m³/ detik)	Conclusion	Explanation
1	Jl. Pramuka Kanan	0,947	1,265	Meets	Uditch 100x100
2	Jl. Pramuka Kiri	0,949	1,265	Meets	Uditch 100x100
3	Jl. Jaksa Agung Suprapto Kanan	0,714	1,265	Meets	Uditch 100x100
4	Jl. Jaksa Agung Suprapto Kiri	0,722	1,265	Meets	Uditch 100x100
5	Jl. Bhayangkara Kanan	0,520	0,698	Meets	Uditch 80x80
6	Jl. Bhayangkara Kiri	0,654	0,698	Meets	Uditch 80x80
7	Jl. Kbp Duriyat Kanan	0,574	0,698	Meets	Uditch 80x80
8	Jl. Kbp Duriyat Kiri	0,652	0,698	Meets	Uditch 80x80

Source: Analysis Results, 2025

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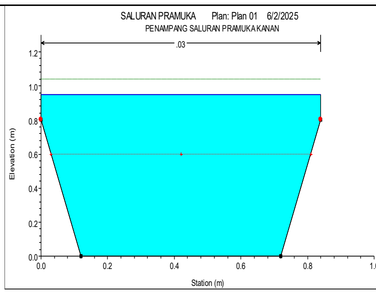
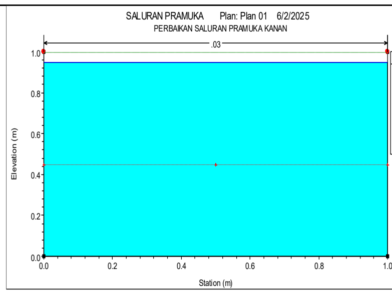
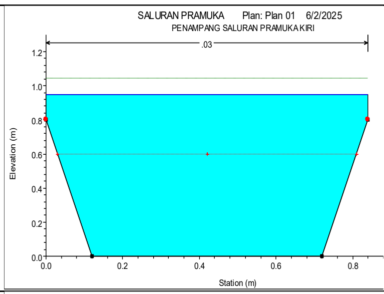
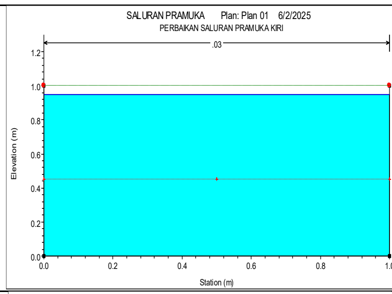
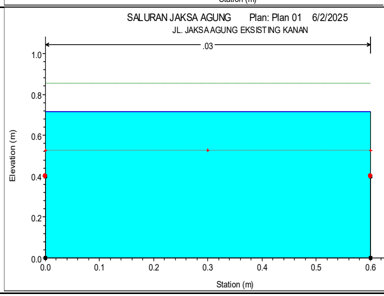
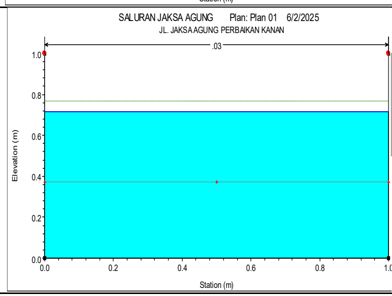
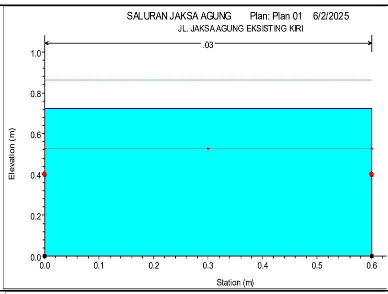
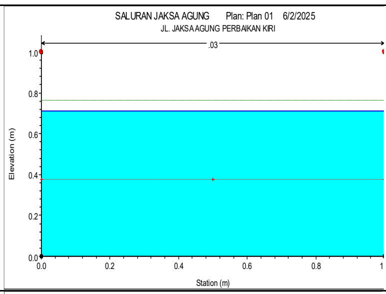
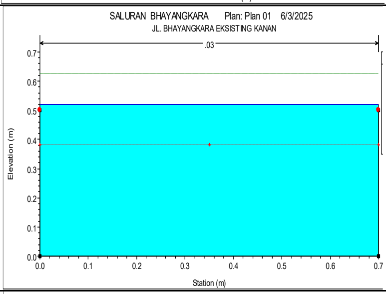
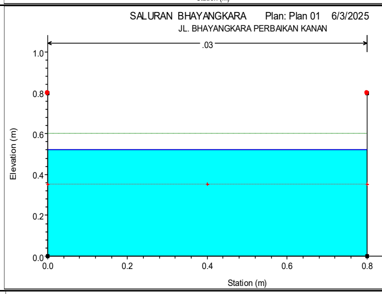
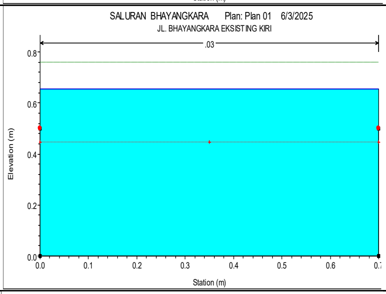
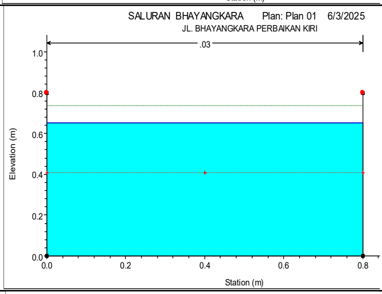
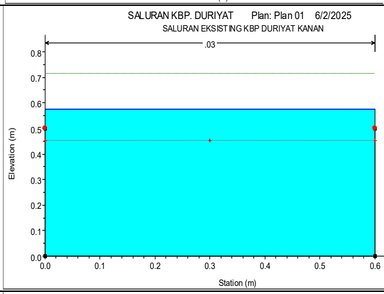
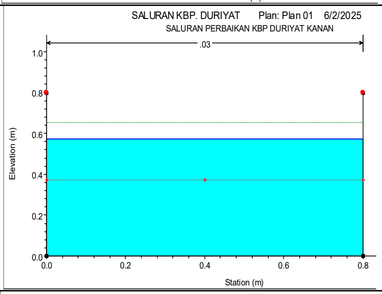
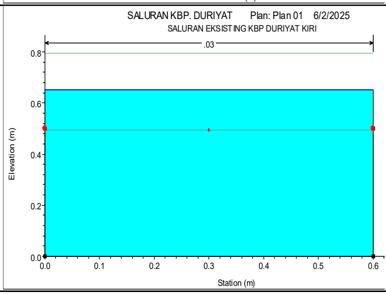
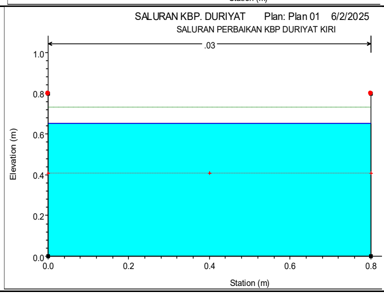
Following a detailed analysis of the relevant data, it was determined that the repairs to the channels on Jl. Pramuka Kanan, Jl. Pramuka Kiri, Jl. Agung Suprapto Kanan, and Jl. Jaksa Agung Suprapto Kiri can be carried out using the Uditch type, which has dimensions of 100 cm x 100 cm. Meanwhile, for Jl. Bhayangkara Kanan, Jl. Bhayangkara Kiri, Jl. Kbp Duriyat Kanan and Jl. Kbp Duriyat Kiri, the Uditch type with dimensions 80 cm x 80 cm should be used.

HEC-RAS

The storage capacity evaluation is a process that is used to determine the existing capacity of the channel before and after the repair. In order to ascertain the storage capacity of the water profile in the drainage channel, the HEC-RAS software was utilized. This software is a computer program developed by the *Hydrologic Engineering Centre* (HEC) to model river and channel flows. The results of the drainage channel water profile are presented in Table 11 below, using the HEC-RAS program:

Table 11. Drainage Water Profile *HEC-RAS* Software

Channel Name	Before Repair	After Repair
Jl. Pramuka Kanan		
Jl. Pramuka Kiri		
Jl. Jaksa Agung Suprapto Kanan		

Channel Name	Before Repair	After Repair
Jl. Jaksa Agung Suprapto Kiri		
Jl. Bhayangkara Kanan		
Jl. Bhayangkara Kiri		
Jl. Kbp. Duriyat Kanan		
Jl. Kbp. Duriyat Kiri		

Source: Analysis Results, 2025

Geographic Information System

The Geographic Information System (*ArcGIS*) is a tool that is utilized for the management, integration, and display of tabular and spatial data on plan maps. The results of hydrological and hydraulic calculations on drainage experiencing overflow will be displayed in a tabular data set on the Regional Spatial Plan Map. The following steps are involved in tabular data modelling in *ArcGIS*:

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Creation of Attribute Table

The creation of an attribute table is instrumental in the display of data pertinent to the research area under analysis.

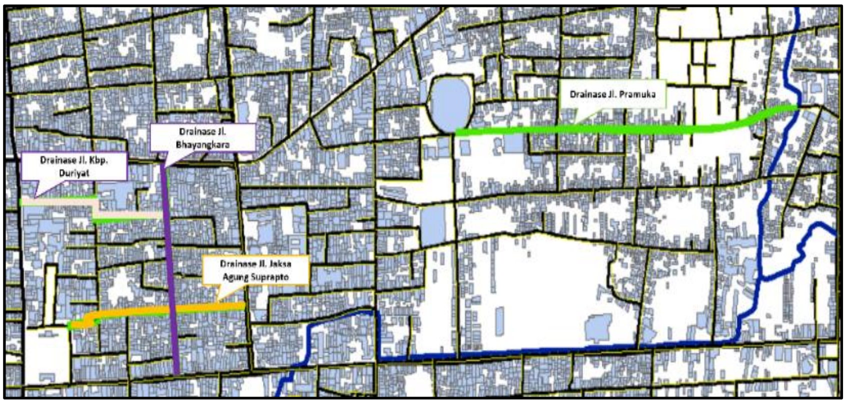

Figure 1. Location of Drainage Study

Source: Analysis Results, 2025

Steps in creating an attribute table by selecting the layer *Saluran Drainase Penelitian-Open Attribute Table*.

Adding a New Field Column

A new field is used to add tabular data when displaying data in *ArcGIS*. To add a *New Field*, select *Table Options-Add Field*. The Field menu will then appear. Figure 2 shows the *ArcGIS* Field menu:

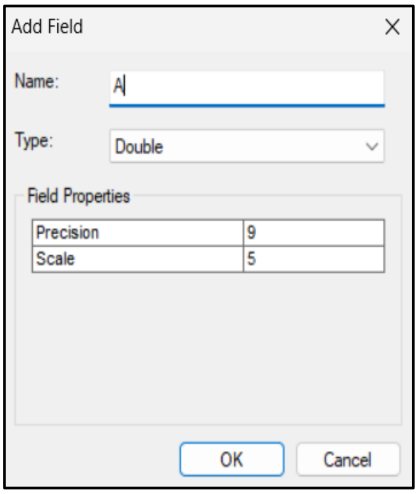

Figure 2. Field Table

Source: Analysis Results, 2025

In the Field menu, a column has been added Luas Lahan (A), Conduction Coefficient (C), Intensities (I), Q_{10}, Q_{total} dan $Q_{tampung}$. For *Type* select type *Double*. Then *Precision* Select 9 and *Scale* Select 5. In Table Q_{10}, it is calculated using the rational discharge equation Q=0,278 C×I×A (in km²). This calculation is calculated by clicking *VB Script*. Then, enter the debit calculation elements based on the *Fields* created. The modelling process is illustrated in Figure 3 below:

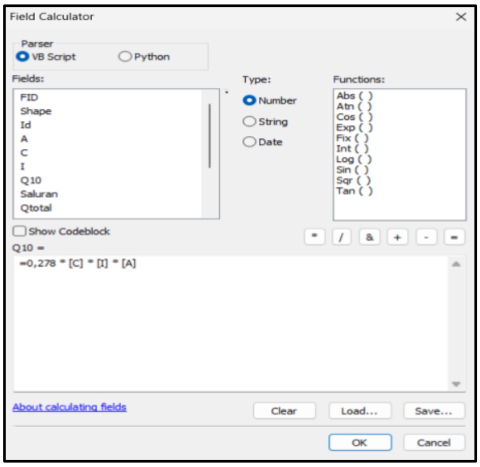

Figure 3. Field Calculator Q_{10}

Source: Analysis Results, 2025

Add Description

In the *Add Field* menu, select Nama: Keterangan and for *Type* select *Text*. After the Description column appears, the next step is to type the letter 'n'. The letter 'n' here serves as a tool to test conditions based on Q_{total} and $Q_{tampung}$ channels. After that, right-click on the description column, select *Field Calculator*. Click *VB Script* and *Show Codeblock*. Then *the Pre-Logic Script Code* field will appear, which can be filled in as shown in Figure 4 below:

dim n
 	
if[Qtotal]>=[Qtampung] Then
n="tidak memenuhi"
 	
elseif[Qtotal]<=[Qtampung] Then
n="memenuhi"
 	
else
n="tidak memenuhi"
end if
	dim n
	
	if[Quditch]>=[Qtotal] Then
	n="ukuran uditch diterima"
	
	elseif[Quditch]<=[Qtotal] Then
	n="ukuran uditch tidak diterima"
	
	else
	n="ukuran uditch tidak diterima"
	end if

Figure 4. Pre-Logic Script Code

Source: Analysis Results, 2025

The *Pre-Logic Script Code* is also filled in the conclusion column. Figure 5 shows the results of creating a drainage channel capacity database:

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Table

Saluran Drainase Penelitian

FID	Shape *	Id	Saluran	A	C	I	Q10	Qtotal	Qtampung	Keterangan	Quditch	Kesimpulan	Ukuran
1	Polyline	0	Jl. Pramuka Kiri	0.13	0.6	42.13	0.94	0.949	0.85	tidak memenuhi	1.265	ukuran uditch diterima	100x100
0	Polyline	0	Jl. Pramuka Kanan	0.12	0.6	42.13	0.87	0.947	0.85	tidak memenuhi	1.265	ukuran uditch diterima	100x100
3	Polyline	0	Jl. Jaksa Agung Kiri	0.06	0.6	72.15	0.72	0.722	0.181	tidak memenuhi	1.265	ukuran uditch diterima	100x100
2	Polyline	0	Jl. Jaksa Agung Kanan	0.05	0.6	72.15	0.66	0.714	0.181	tidak memenuhi	1.265	ukuran uditch diterima	100x100
5	Polyline	0	Jl. Bhayangkara Kiri	0.05	0.6	62.28	0.60	0.654	0.216	tidak memenuhi	0.698	ukuran uditch diterima	80x80
7	Polyline	0	Jl. Kbp. Duriyat Kiri	0.05	0.6	78.15	0.65	0.652	0.253	tidak memenuhi	0.698	ukuran uditch diterima	80x80
6	Polyline	0	Jl. Kbp. Duriyat kanan	0.04	0.6	78.15	0.57	0.574	0.253	tidak memenuhi	0.698	ukuran uditch diterima	80x80
4	Polyline	0	Jl. Bhayangkara Kanan	0.05	0.6	62.28	0.51	0.52	0.216	tidak memenuhi	0.698	ukuran uditch diterima	80x80

Figure 5. *ArcGIS* Tabular Table

Source: Analysis Results, 2025

Conclusion

The results of the 10-year flood discharge analysis, with rational calculations, were as follows: *Jl. Pramuka Kanan* at 0.879 m³/s, *Jl. Pramuka Kiri* at 0.949 m³/s, *Jl. Jaksa Agung Suprapto Kanan* at 0.662 m³/s, *Jl. Suprapto Kiri* at 0.722 m³/s, *Jalan Bhayangkara Kanan* at 0.520 m³/s, *Jalan Bhayangkara Kiri* at 0.654 m³/s, *Jalan Kbp Duriyat Kanan* at 0.574 m³/s, and *Jalan Kbp Duriyat Kiri* at 0.652 m³/s. The calculation results indicate that the 10-year return period discharge in the existing drainage channels exceeds their capacity, necessitating repairs. The repairs were executed using *U-Ditch* 100 cm × 100 cm for *Jl. Pramuka Kanan*, *Jl. Pramuka Kiri*, *Jl. Jaksa Agung Suprapto Kanan*, and *Jl. Jaksa Agung Suprapto Kiri*. In addition to the enhancements made by utilizing *U-Ditch* 80 cm × 80 cm for *Jl. Bhayangkara Kanan*, *Jl. Bhayangkara Kiri*, *Jl. KBP Duriyat Kanan*, and *Jl. KBP Duriyat Kiri*, further developments are anticipated.

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