Vol. 13, No. 4 (2024): 1023 - 1035 http://dx.doi.org/10.23960/jtep-l.v13i4.1023-1035

JURNAL TEKNIK PERTANIAN LAMPUNG

ISSN 2302-559X (print) / 2549-0818 (online)
Journal homepage : https://jurnal.fp.unila.ac.id/index.php/JTP

Avoiding Flood by Improving Cross-Sectional Capacity through River Normalization

Muzhaffar Harits[1,✉], Asep Sapei[1], Nora Herdiana Pandjaitan[1]

[1] Departement of Civil and Enviromental Enginerring, Fakulty of Agricultural Technology, IPB University, Bogor, INDONESIA.

Article History:

Received : 15 June 2024
Revised : 07 August 2024
Accepted : 09 August 2024

Keywords:

Flood,
HEC-RAS,
Nakayasu,
Normalization,
Rainfall.

Corresponding Author:
✉ haritsmuzhaffar@apps.ipb.ac.id
(Muzhaffar Harits)

ABSTRACT

Bekasi is one of the regions in Indonesia that often suffer from flooding. To overcome flooding problems in the Bekasi area, the Ciliwung Cisadane River Basin Center has begun a normalization project along the Bekasi River. This research aimed to evaluate the Bekasi River's cross-sectional capacity both before and after normalization. The analysis focused on the section where the Cileungsi and Cikeas rivers converge to the Bekasi weir. In this research, secondary data were employed, such as national digital elevation model data, detailed engineering designs of Bekasi River flood control activities, and rainfall data from the Cibinong Station that covers the period from 2006 to 2020. The result from planned flood discharge with the Nakayasu synthetic unit hydrograph method for a return period of 25 years on the Bekasi River was obtained at 665.81 m³/s. The results of hydraulic analysis before normalization with the HEC-RAS application show that flooding occurred at 102 out of 116 stations, with capacities of river cross-sections ranging from 453.49 m³/s to 665.71 m³/s at these overflow stations. Following the river normalization process, the cross-sectional capacity at all Bekasi River stations can accommodate flood discharge without any instances of overflow.

1. INTRODUCTION

Flooding is an event that occurs when a channel or river overflows, resulting in water inundating the surrounding land. The factors contributing to floods can be categorized into two types: human factors and natural factors. Human factors include deforestation, which increases water runoff; improper urban planning, which leads to reduced catchment areas; and pollution of waterways, thereby reducing their capacity. Natural factors encompass physiographic influences, rainfall patterns, sedimentation, erosion, and river capacity (Kodoatie & Sugiyanto, 2002). Flood disasters frequently occurred in Indonesia, leading to significant material and immaterial losses. One of the worst impacts of the flood disaster, which affects the socio-economic conditions of the community, is a decline in health, a decrease in income and obstacles in daily activities (Yunida *et al*., 2017).

One of the region in Indonesia that often suffer from flooding is Bekasi. With the increasing demand for housing due to population growth in the Jabodetabek (Jakarta-Bogor-Depok-Tangerang-Bekasi) area, there had been a reduction in urban green space within the Bekasi Hulu Watershed. This results in a reduction in infiltration flow while it increases surface flow (Djunit & Aziz, 2020). Furthermore, the Bekasi River, the main river of Bekasi City, accommodates flows from the Cileungsi River and Cikeas River. During periods of heavy rainfall, the Bekasi River receives a significant discharge of water, leading to flooding in areas surrounding the Bekasi River, including the Villa Nusa Indah, Pondok

Gede Permai, Kemang Ifi Graha, Jatiasih Indah, Pondok Mitra Lestari, Jaka Kencana, Kemang Pratama, and other areas (Rojali & Elsari, 2020). This situation was further exacerbated by erosion and sedimentation, which resulted in a decreased cross-sectional capacity of the Bekasi River.

To overcome flooding problems in the Bekasi area, the Ciliwung Cisadane River Basin Center has begun a normalization project along the Bekasi River. River normalization is an activity to improve and restore the normal function of the river itself while simultaneously addressing flood problems in the areas surrounding the river. The activity may include planning and constructing new river cross-sections and levees with appropriate designs and reinforcing riverbanks. These activities aim to increase the Bekasi River's Capacity to accommodate the planned flood discharge and to strengthen the river's structural stability, thereby reducing erosion that diminishes the river's capacity. Consequently, it became crucial to analyze the impact of these flood control activities on the Bekasi River, specifically focusing on the section where the Cileungsi River and Cikeas River converge to the Bekasi weir. This research aimed to asses the Bekasi River's cross-sectional capacity before and after normalization procces, especially in relation to flooding potential.

2. RESEARCH METHODS

2.1. Location and Research Time

This research took place from March to July 2023. The analysis focused on the section where the Cikeas and Cileungsi Rivers converge to the Bekasi Weir. Data processing occurred at the Department of Civil and Environmental Engineering, Faculty of Agricultural Technology, IPB. The research location map is presented in the Figure 1.

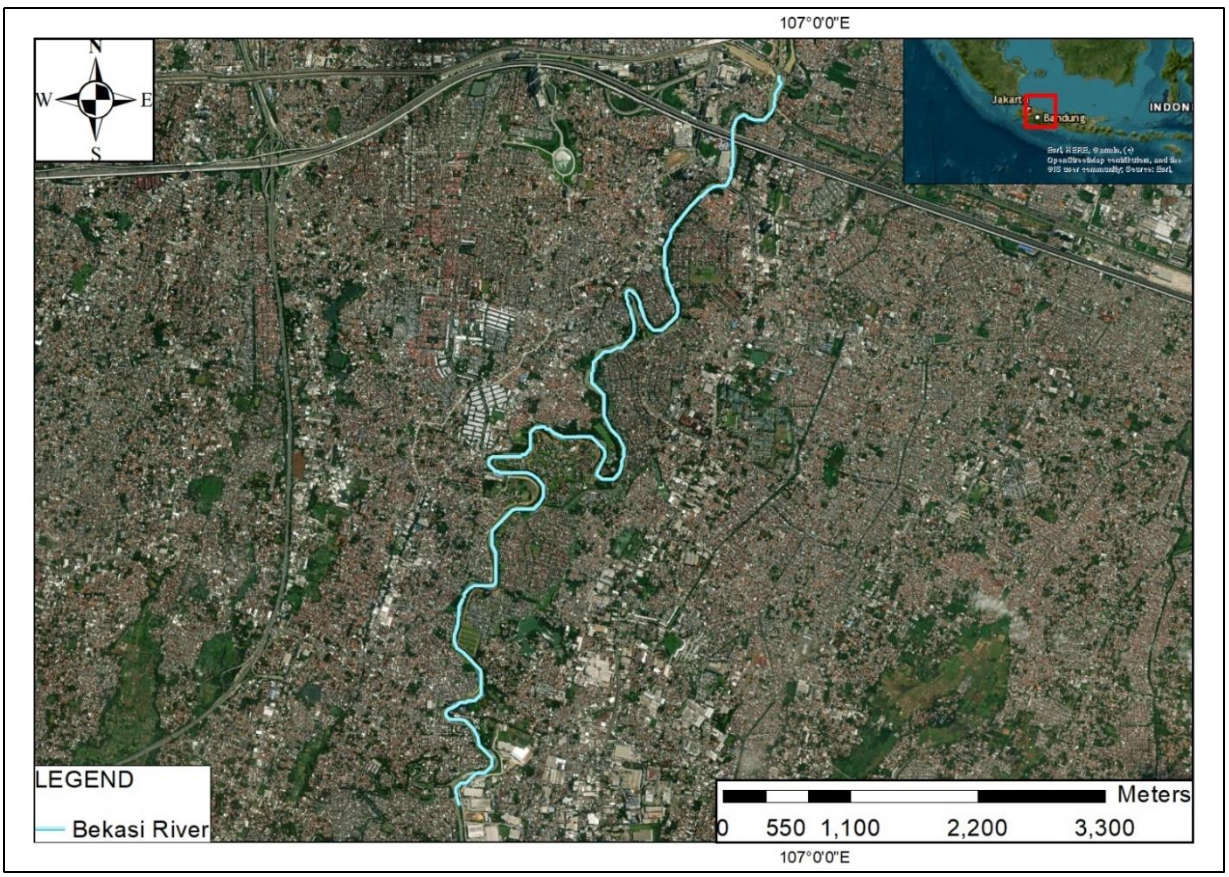

Figure 1. Research location map

2.2. Equipment and Materials

The equipment utilised included a laptop equipped with MS Excel software for processing planning data, ArcMap for creating research location maps, and HEC-RAS for modelling flow in rivers. Primary data were obtained through discussions and interviews with relevant stakeholders. Secondary data, such as national digital elevation model data, detailed engineering designs of Bekasi River flood control activities, and rainfall data, were employed.

2.3. Research Framework

The research process comprised several stages, including literature review and data collection, data processing, and river flow modelling. The research began with a literature study and data collection. The literature study aimed to understand the stages of river flow modelling using HEC-RAS software. This was done by reading various books, journals, and related reports. Data processing included planned daily rainfall analysis, planned flood discharge analysis, and hydraulic cross-sections of the river analysis.

Planned rainfall analysis commenced with analysing maximum daily rainfall frequency, considering statistical parameters such as standard deviation, coefficient of variation, coefficient of kurtosis, and coefficient of skewness. Standard deviation measures the extent of variability of a variable's values around its mean, and it can be calculated using Equation (1). The coefficient of variation measures the dispersion of data points around the mean in a dataset, and it can be determined using Equation (2).

After that, the coefficient of kurtosis is intended to measure the sharpness of the shape of the distribution curve, and generally compared to a normal distribution. Equation (3) can be used to calculate the coefficient of kurtosis (Ardiansyah *et al*., 2021). The coefficient of skewness represents the degree of asymmetry in a distribution. The equation to calculate the coefficient of skewness is given in Equation (4) (Ruhiat, 2022).

$$S_d = \sqrt{\frac{\Sigma(X_i - X_{rt})^2}{n-1}} \tag{1}$$

$$C_v = \frac{S_d}{X_{rt}} \tag{2}$$

$$C_k = \frac{n^2\,\Sigma(X_i - X_{rt})^4}{(n-1)(n-2)(n-3)\,{S_d}^4} \tag{3}$$

$$C_s = \frac{n\,\Sigma(X_i - X_{rt})^3}{(n-1)(n-2)\,{S_d}^3} \tag{4}$$

where X_i is maximum rainfall height for n years (mm), X_{rt} is average maximum daily rainfall for n years (mm), S_d is standard deviation, C_v is coefficient of variation, C_k is coefficient of kurtosis, C_s is coefficient of skewness, and n is number of rainfall data.

The slope coefficient and coefficient of kurtosis values obtained from the calculations were then compared with the standard slope coefficient and coefficient of kurtosis values for the applicable distribution method. Following this, the planned rainfall was determined using the distribution method, which had slope coefficient and coefficient of kurtosis values that met the standard criteria. Four distribution methods can be used to analyse planned rainfall: the Gumbel method, the normal method, the log Pearson type III method, and the log normal method. Planned rainfall using Gumbel and normal methods can be computed using Equation (5) and Equation (6) (Soemarto, 1999), whereas Equation (7) and Equation (8) are utilised for log Pearson type III and log normal methods.

$$X_T = \overline{X} + \frac{S_d}{S_n}(Y_t - Y_n) \tag{5}$$

$$X_T = \overline{X} + K_T \text{ x } S_n \tag{6}$$

$$\text{Log } X_t = \sqrt{\text{Log } X_{rt}} + K * S_d \tag{7}$$

$$\text{Log } X_t = \sqrt{\text{Log } X_{rt}} + K_T * S_d \tag{8}$$

where Y_t is reduced variate at a return period of t years, Y_n is average reduced mean based on the amount of data (n), S_n is reduced standard deviation based on the amount of data (n), S is standard deviation, X is average rainfall (mm), X_T is planned rainfall (m^3/s), K_T is gauss reduction variable for the return period T years, K is log Pearson type III variable, Log X_{rt} is the Logarithmic average rainfall, Log X_t is the Logarithmic planned rainfall.

Then, the suitability of the distribution is evaluated with the chi-square test and the Smirnov-Kolmogorov test. This test provides insight into how well the observed data aligns with the distribution model under examination (Husna *et al*., 2022). The chi-square test evaluates the disparity between the observed and expected frequencies, subsequently assessing the extent to which this disparity can be attributed to random factors or significant differences. The calculation of the observed frequency in the chi-square test can be performed using Equation (9), Equation (10), Equation (11), and Equation (12) (Upomo & Kusumawardani, 2016). Meanwhile, the Smirnov-Kolmogorov test compares the cumulative distribution function of the expected distribution with the empirical distribution function of the observed data. In the Smirnov-Kolmogorov test, the observed distribution value can be calculated using Equation (13) (Bagaskara *et al*., 2024).

$$G = 1 + 3.32 \text{ Log } n \tag{9}$$

$$Dk = G - (R + 1) \tag{10}$$

$$E_i = \frac{n}{G} \tag{11}$$

$$x^2 = \Sigma \frac{(O_i - E_i)^2}{E_i} \tag{12}$$

$$Dmaks = P(Xn) - P'(Xn) \tag{13}$$

where G is number of subgroups, n is number of data, Dk is observed frequency distribution, R is 2 for normal and binomial distribution, 1 for poison, E_i is expected value, O_i is observed value, x^2 is observed frequency for chi-square test, Dmaks is observed frequency for Smirnov-Kolmogorov test, P(Xn) is observed probability, and P'(Xn) is theoretical probability.

The peak runoff (flood) discharge magnitude can be estimated using the modified rational method. This method is used a development of the rational method for rainfall intensity that is longer than the concentration time. Theoretical analysis of flood discharge can be carried out using rational methods. The equation used to determine the peak discharge for the rational method is Equation (14) (Feyen, 1980; Herison *et al*., 2018).

$$Q = 0.00278\, C * I * A \tag{14}$$

$$I = \frac{R24}{24} \left[\frac{24}{T_c}\right]^{\frac{2}{3}} \tag{15}$$

$$T_c = \left(\frac{0.87 * L^2}{1000 * S}\right)^{0.385} \tag{16}$$

where A represents the watershed area (ha), C is the runoff coefficient, I denotes the rainfall intensity during concentration time (mm/hour), Q stands the planned flood discharge (m^3/s), R24 is planned rain in one day (mm), Tc indicates rain concentration time (hours), S is river slope (%), and L is river length (km)

Theoretical analysis of runoff discharge also requires a rainfall intensity value. Rainfall intensity refers to the ratio of the total amount of rain falling during a given period to the duration of the period. The general characteristic of rainfall is that the shorter its duration, the greater its intensity, and the longer the return period, the higher its intensity. Rainfall intensity is obtained by statistically and empirically analysing rain data. Rainfall intensity is typically associated with short-term rain duration data, such as 5 minutes, 30 minutes, 60 minutes, and hours. If short term rain duration data is unavailable and there is only daily rain data, the rainfall intensity can be estimated with the Mononobe formula, as shown in Equation (15) (Triatmodjo, 2010).

Rain concentration time refers to the period it takes for rainwater to travel from the furthest point to a specific location within the drainage area (Suripin, 2004). The concentration time typically includes the period required for water to move across the ground surface to the closest river or channel and the period to flow within the river or channel until it reaches the specified location. An approximation of the concentration time can be obtained using Equation (16).

The runoff coefficient is an indicator of whether a watershed has experienced disturbance. A large C value indicates that much rainwater becomes runoff. Surface runoff coefficient is the ratio or ratio between surface flow and rainfall that falls in a water catchment area. The surface flow coefficient value can describe the criticality of a water catchment area, especially regarding its hydrological conditions (Krisnayanti *et al.*, 2018).

Additionaly, the planned flood discharge also can be estimated through the Nakayasu synthetic unit hydrograph method with various return periods. The Nakayasu method was used in this study due to its high accuracy, ease of accessing the required data, ability to handle different rainfall variations, including extreme rainfall, and applicability to various types of watersheds (Damayanti *et al.*, 2022). This method is based on the principle that surface flow is influenced by the physical characteristics and topography of the area, and the rainfall that occurs in a specific period. This method uses intensity-duration-frequency (IDF) rainfall data and regional characteristics such as area, average rainfall, flow coefficient, and concentration time to produce a synthetic unit hydrograph (Margini *et al.*, 2017). This method is commonly used in hydrological planning and analysis to estimate surface flow in various rainfall scenarios. Equation (17) is used to estimate the planned flood discharge using the Nakayasu synthetic unit hydropraph method.

$$\mathrm{Q_p} = \frac{\mathrm{A} * \mathrm{R}}{3.6\left(0.3\,\mathrm{T_p} + \mathrm{T_{0.3}}\right)} \tag{17}$$

$$\mathrm{T_g} = 0.21\,\mathrm{L}^{0.7} \text{ for } \mathrm{L} < 15\text{ km} \tag{18}$$

$$\mathrm{T_g} = 0.4 + 0.058\,\mathrm{L} \text{ for } \mathrm{L} > 15\text{ km} \tag{19}$$

$$\mathrm{T_r} = 0.75\ \mathrm{T_g} \tag{20}$$

$$\mathrm{T_p} = \mathrm{T_g} + 0.8\,\mathrm{T_r} \tag{21}$$

$$\mathrm{T_{0.3}} = \alpha \text{ x } \mathrm{T}_r \tag{22}$$

$$\alpha = \frac{0.47\text{ A x L}^{0.25}}{\mathrm{T_g}} \tag{23}$$

$$\mathrm{Q_t} = \mathrm{Q_p}\left(\frac{\mathrm{t}}{\mathrm{T_p}}\right)^{2.4} \text{ for } 0 < \mathrm{t} \leq \mathrm{T_p} \tag{24}$$

$$\mathrm{Q_t} = 0.3\,\mathrm{Q_p}^{\left(\frac{\mathrm{t-Tp}}{\mathrm{T_{0.3}}}\right)} \text{ for } \mathrm{T_p} < \mathrm{t} \leq (\mathrm{T_p} + \mathrm{T_{0.3}}) \tag{25}$$

$$\mathrm{Q_t} = 0{,}3\,\mathrm{Q_p}^{\left(\frac{\mathrm{t-T_p}+0{,}5\,\mathrm{T_{0,3}}}{1{,}5\,\mathrm{T_{0,3}}}\right)} \text{ for } (\mathrm{T_p} + \mathrm{T_{0.3}}) \leq \mathrm{t} \leq (\mathrm{T_p} + \mathrm{T_{0.3}} + 1.5\,\mathrm{T_{0.3}}) \tag{26}$$

$$\mathrm{Q_t} = 0.3\,\mathrm{Q_p}^{\left(\frac{\mathrm{t} - \mathrm{T_p} + 1.5\,\mathrm{T_{0.3}}}{2\,\mathrm{T_{0.3}}}\right)} \text{ for } \mathrm{t} \geq (\mathrm{T_p} + \mathrm{T_{0.3}} + 1.5\,\mathrm{T_{0.3}}) \tag{27}$$

where R represents the unit rain (mm), A denotes the watershed area (km^2), Q_p is peak flood discharge (m^3/s), T_p is the peak flood time (hours), T_g is time period from rain to flood (hours), T_r is unit of rain duration (hours), α is watershed characteristics, Q_t is discharge at the time *t* (m^3/s), and $T_{0.3}$ (h) signifies the period for the Q_p value to decrease to $0.3Q_p$. T_g is the time lag or the time between rain and peak flood discharge (hours) and is calculated based on the river channel length provisions as in Equation (18) and Equation (19). To calculate T_r, the formula uses Equation (20), while T_p is calculated using Equation (21). $T_{0.3}$, or the time for the Q_p value to decrease to 0.3 Q_p, is calculated using Equation (22), and watershed characteristics are determined using Equation (23). The formulas used to describe the Nakayasu hydrograph are Equation (24), Equation (25), Equation (26), and Equation (27) (Andayani & Umari, 2022).

The collected data is utilised for flow and flood modelling using the HEC-RAS application. The initial step involves inputting flood discharge data and specifying the type of river flow. Subsequently, longitudinal section data of the river and corresponding cross-section data are entered based on station numbers (sta), assuming smaller station numbers indicate closer proximity to the river mouth. The longitudinal and cross-section data comprise distance and ground elevation values. Additionally, x-axis data for the left and right banks, river slope value, and Manning coefficient value are required. The outcome of running the HEC-RAS application includes modelling river cross-sections with floodwater level heights and one-dimensional flow analysis with hydrograph input. The HEC-RAS program's results are crucial in this research, particularly for determining appropriate embankment elevations based on land conditions.

3. RESULTS AND DISCUSSION

3.1. Research Location

The research area is located within the upper Bekasi watershed, which geographically is at 06° 14' 09" - 06° 42' 21" South Latitude and 106° 49' 05" - 107° 01' 47" East Longitude. The Bekasi Hulu River is part of the Bekasi River, which originates from the Cileungsi and Cikeas Rivers and flows north of Bekasi City. The location of the upstream Bekasi watershed is presented in Figure 2. The Bekasi Hulu watershed exhibits a parallel flow pattern characterized by two large rivers in its downstream area. Administratively, the Upper Bekasi Watershed spans Bekasi Regency, Bekasi City, Bogor Regency and within West Java Province. Covering an area of 392.568 km^2, the Bekasi Hulu Watershed is subdivided into the Cileungsi Sub-watershed and the Cikeas Sub-watershed. Within the Bekasi Hulu watershed, three

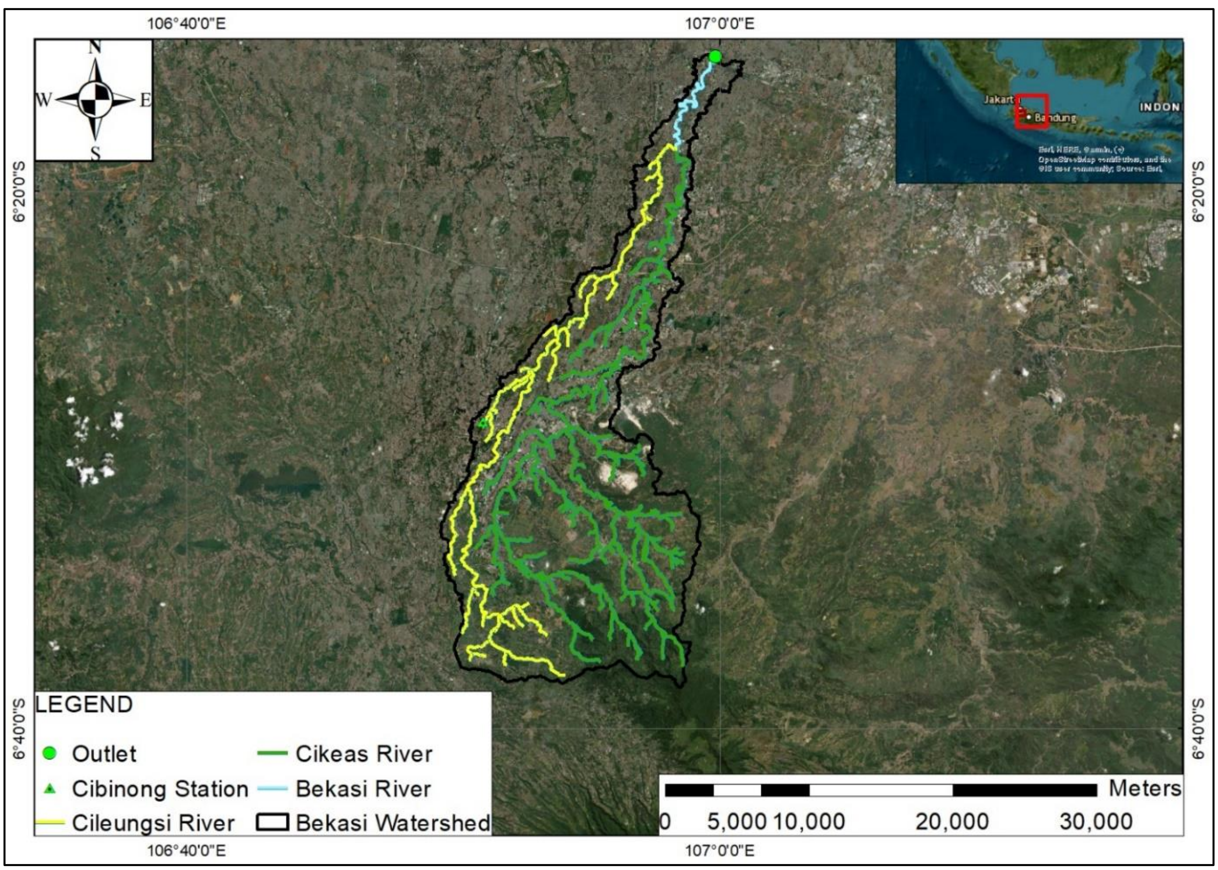

Figure 2. Bekasi Hulu watershed location

main rivers are identified: the Bekasi Hulu River, spanning 11.57 km in length; the Cikeas River, with a length of 49.924 km; and the Cileungsi River, which stretches 50.67 km in length. Consequently, the total river length within the Bekasi Hulu watershed amounts to 112.17 km

3.2. Planned Rainfall Analysis

Rainfall analysis was conducted using maximum daily rainfall data for the last 15 years acquired from the nearest Meteorology, Climatology, and Geophysics Agency Statio, namely Cibinong Station. The Cibinong Station Data used for this research covers the period from 2006 to 2020. Table 1 presents the maximum daily rainfall values. The highest peak rainfall value was recorded in 2006, reaching 150 mm, while the lowest was in 2015, measuring 70 mm. After obtaining the maximum daily rainfall, frequency analysis was conducted to estimate planned rainfall. Four types of frequency distribution methods were employed to determine planned rainfall: the Gumbel method, normal method, log Pearson type III method, and log normal method. Table 2 are summarized the results of the planned rainfall analysis using these four frequency distribution methods.

Table 1. Maximum daily rainfall value

Year	Max daily rainfall (mm)	Year	Max daily rainfall (mm)	Year	Max daily rainfall (mm)
2006	150.0	2011	90.0	2016	135.0
2007	80.0	2012	89.0	2017	85.0
2008	132.0	2013	82.0	2018	100.0
2009	77.0	2014	81.0	2019	107.5
2010	75.0	2015	70.0	2020	104.0

Table 2. Planned rainfall

Return period	Planned rainfall (mm/day)			
	Gumbel	Normal	Log Pearson type III	Log Normal
2	93.68	97.17	91.80	94.63
5	120.70	117.60	113.45	115.02
10	138.58	128.31	129.06	127.40
25	161.19	138.73	150.30	140.73
50	177.95	147.04	167.20	152.36

Table 3. Distribution type criteria

Distribution type	Criteria	Result
Gumbel	$C_k \approx$ 5.40	3.68
	$C_s \approx$ 1.14	1.09
Normal	$C_k \approx$ 3.00	3.68
	$C_s \approx$ 0.00	1.09
Log normal	$C_s \approx 3C_v \approx$ 3.00	0.79
Log Pearson type III	If the others type is not match	

The frequency distribution method for calculating planned rainfall is selected and determined through statistical parameter analysis. These statistical parameters contain standard deviation, coefficient of variation, coefficient of kurtosis, and coefficient of skewness. Selecting an inappropriate frequency distribution method may lead to significant estimation errors. Based on the analysis results presented in Table 3, the distribution method chosen for this research is the log Pearson type III. This decision was made because the other three distribution methods failed to meet the required criteria. Conversely, the log Pearson type III method offers flexibility by not imposing limitations on the coefficient of variance or coefficient of kurtosis.

In estimating planned flooding using distribution analysis, it is necessary to carry out distribution suitability tests.

These tests ensures that the empirical approach accurately represents the theoretical curve. The chi-square and the Smirnov-Kolmogorov tests can be utilized for this purpose (Limantara, 2010). In the chi-square test, the expected frequency (x^2 crit) value must exceed the observed frequency value (x^2 calc) (Soewarno, 1995). With confidence interval and degrees of freedom values set at 5% and two, respectively, the expected frequency value is 5.991, while the expected frequency value is 1.33. Therefore, the observed frequency value meets the requisite criteria. The results of the chi-square test are shown in Table 4.

Table 4. Chi-square test results

Interval	Ei	Oi	Oi - Ei	$(Oi-Ei)^2/Ei$
> 113.45	3.00	2.00	-1.00	0.33
95.09 – 113.45	3.00	2.00	-1.00	0.33
85.18 – 95.09	3.00	3.00	0.00	0.00
77.57 – 85.18	3.00	2.00	-1.00	0.33
< 77.57	3.00	2.00	-1.00	0.33
Total	15.00	11.00	x^2	1.33

Table 5. Smirnov-Kolmogorov test results

Xi	Log Xi	P(x)	P'(x)	D
150.00	2.18	0.063	0.024	0.039
135.00	2.13	0.125	0.063	0.062
132.00	2.12	0.188	0.076	0.111
107.50	2.03	0.250	0.291	0.041
104.00	2.02	0.313	0.323	0.010
100.00	2.00	0.375	0.405	0.030
90.00	1.95	0.438	0.587	0.150
89.00	1.95	0.500	0.595	0.095
85.00	1.93	0.563	0.677	0.115
82.00	1.91	0.625	0.732	0.107
81.00	1.91	0.688	0.749	0.061
80.00	1.90	0.750	0.764	0.014
77.00	1.89	0.813	0.813	0.001
75.00	1.88	0.875	0.841	0.034
70.00	1.85	0.938	0.903	0.034
			Dmaks	0.150

Table 5 is presented the result of the Smirnov Kolmogorov test. The observed frequency distribution (Dmaks) value obtained is 0.15, while the expected frequency distribution (Dcrit) value is 0.409, known using significant degree data of 0.05. The observed frequency distribution value must be smaller than the expected frequency distribution for the Smirnov Kolmogorov method. Consequently, the observed frequency distribution value satisfies the requirements of the Smirnov Kolmogorov test. It is concluded that the log Pearson type III distribution method is suitable for calculating planned rainfall in this location.

3.3. Planned Flood Discharge Analysis

Planned flood discharge analysis is heavily contingent upon the availability of maximum instantaneous flood discharge data and the length of the observation period. When such data are scarce or unavailable, planned rainfall data can provide an alternative for estimating planned flood discharge using the empirical relationship between rainfall and runoff. The methods that can be used for this analysis are the rational method and the Nakayasu synthetic unit hydrograph method.

Planned flood discharge rational method involves determining parameters such as the runoff coefficient, catchment area, and rainfall intensity. The planned rainfall data utilized in this study was derived from analysis with log Pearson type III. The rational method provides a practical means to establish the relationship between runoff discharge and rainfall, typically suitable for watersheds up to 5000 ha (BSN, 2016). However, given that the area of the Bekasi Hulu watershed exceeds this threshold, the rational methods cannot be used to calculate the planned flood discharge. The Bekasi Hulu watershed spans 392.57 km^2 and has a total river length of 112.17 km. Consequently, it exhibits a lag time of 6.906 hours and a peak time of 11.049 hours. Time lag refers to the delay between the peak rainfall and the peak discharge in a river, while peak time denotes the duration for the discharge to reach its maximum level (Suripin, 2004). The peak hydrograph discharge is determined to be 10.772 m^3 based on the obtained peak value/sec.

The planned flood discharge Nakayasu synthetic unit hydrograph method was analysed for 2, 5, 10, 25, and 50-year return periods. As depicted in Figure 3, the discharge and time values consistently increase until 11.049 hours, after which they decrease. The Nakayasu flood hydrograph illustrates the discharge progression from the onset of rainfall through the flood to its conclusion. Considering the city typology, a return period of 25 years was chosen since the Bekasi Hulu watershed falls within the metropolitan area, encompassing an area exceeding 500 ha. The planned flood discharge for the Bekasi River was obtained at 665.81 m^3/s.

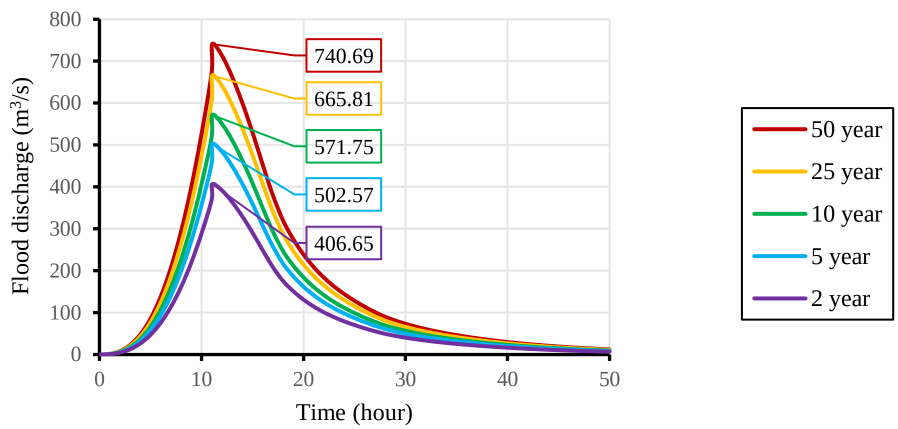

Figure 3. Nakayasu synthetic unit hydrograph

3.4. Hydraulic Cross-Section of the River Analysis

Cross-sectional river capacity was determined through one-dimensional hydraulic modelling with the HEC-RAS program. Cross-sectional data on the Bekasi River before and after normalization project was sourced from the results of the Bekasi River Flood Control Project Package 1 Detailed Engineering Design (DED) provided by the Ciliwung Cisadane River Basin Center. Longitudinal cross-section data of the river are delineated every 0.1 km, spanning from the upstream section (station number 115) to the downstream section (station number 0), totaling 116 stations. The slope value of the Bekasi River was calculated by comparing the height difference with the river's length, resulting in 0.0033. Additionally, the Manning roughness value for the Bekasi River was determined to be 0.033, considering it is a natural channel. The results of the Bekasi River's cross-sectional capacity analysis before normalization, conducted using HEC-RAS with the flood water level, are depicted in Figure 4.

The embankments along the Bekasi River vary in height from 2 to 4 m, but certain river sections remain without embankments. Analysis indicates overflow occurred at 102 out of 116 stations along the river. Consequently, measures must be taken to address the overflow. At these 102 overflow stations, the capacities of the river cross- sections range from 453.49 m³/s to 665.71 m³/s, which was insufficient to accommodate the 665.81 m^3/s flood discharge. The flood water level resulting from the Bekasi River flood discharge ranges from 18.68 to 27.19 m above sea level (masl).

River normalization was implemented through channel dredging and embankment construction to mitigate flooding in the Bekasi River. Following normalization, the slope value of the Bekasi River changed to 0.0063. In contrast, the Manning roughness value decreased to 0.025 due to the excavation of the river channel, resulting in reduced river depth and improved water quality. After normalization, the capacities of the river cross-sections range from 692.52 m³/s to 884.91 m³/s. The flood water level resulting from the Bekasi River flood discharge ranges from 15.27 masl to 24.92 masl. This means a reduction of flood water level of 1.96 m to 3.62 m as compared to those of before normalisation.

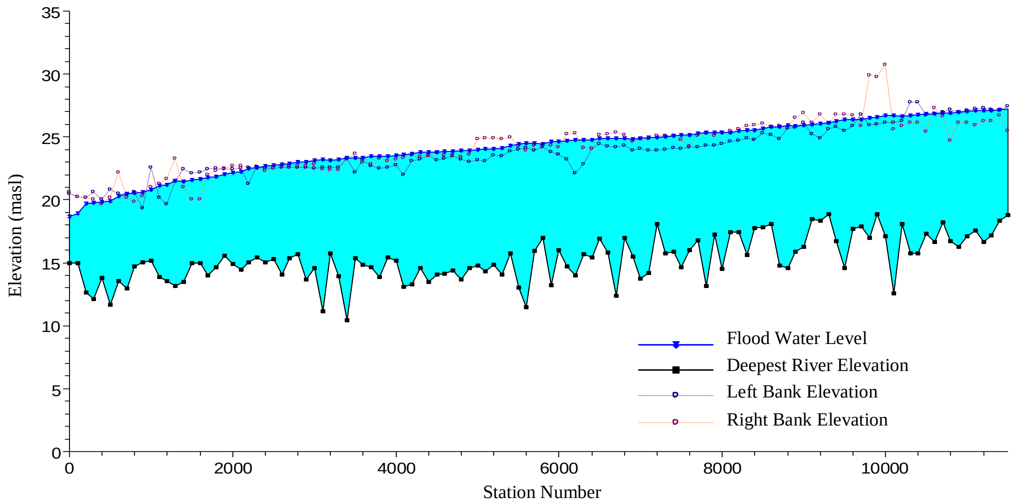

Figure 4. Longitudinal section of the Bekasi River before normalization

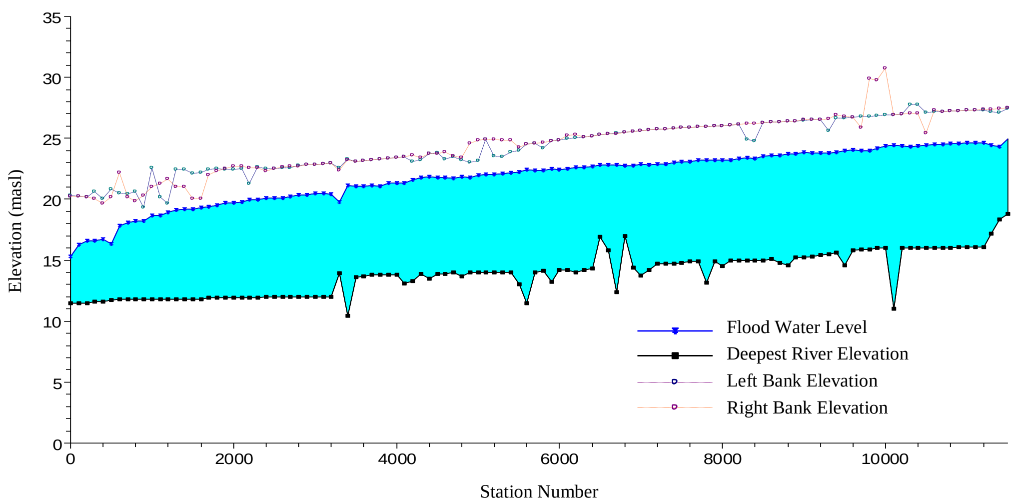

Figure 5. Longitudinal section of the Bekasi River after normalization

The analysis results of the Bekasi River's cross-sectional capacity after normalization, conducted using HEC-RAS with flood water level for a return period of 25 years can be seen in Figure 5. Dredging of the river was not applied at all stations. Dredging was only carried out when the river could not accommodate the planned flood discharge after the construction of new levees. Consequently, the deepest river elevation did not decrease consistently. There were fluctuations in the deepest river elevation, as they followed the natural cross-section of the river, such as at stations 6900 to 6300. The analysis indicates that the cross-sectional capacity at all Bekasi River stations is sufficient to accommodate flood discharge, with no instances of overflow. The increased cross-sectional capacity of the Bekasi River is attributed to the larger cross-sectional volume resulting from riverbed dredging and embankment construction, coupled with faster water flow due to improved river cleanliness after normalization. Consequently, the normalization efforts have effectively mitigated flooding in the Bekasi River.

Figure 6 depicts one of the stations along the Bekasi River, station 1100, which experienced overflow and lacks embankments on both sides. The left cliff of station 1100 has an elevation of 20.160 masl, while the right cliff stands at 21.253 masl. The flood water level of the Bekasi River at station 1100 reaches 21.15 masl, causing the river's left bank to experience overflow.

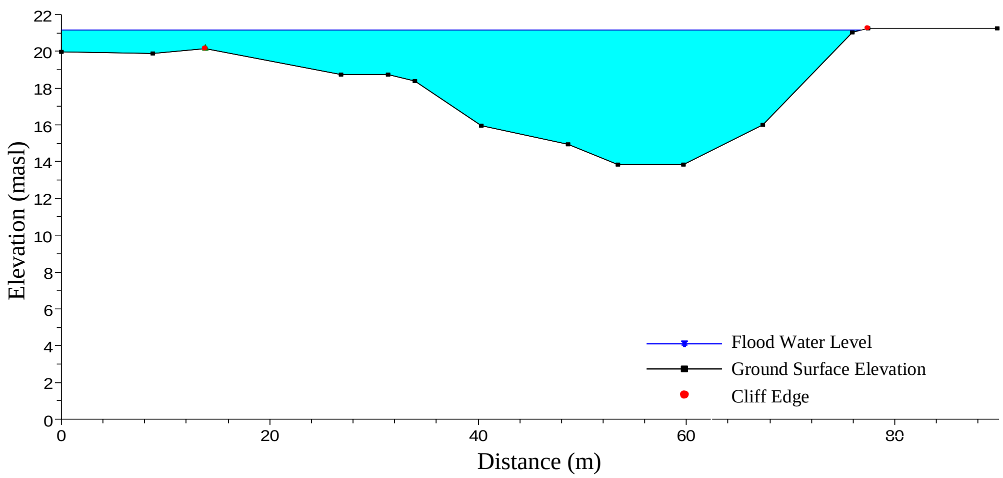

Figure 6. Cross-section of the Bekasi River at station 1100 before normalization

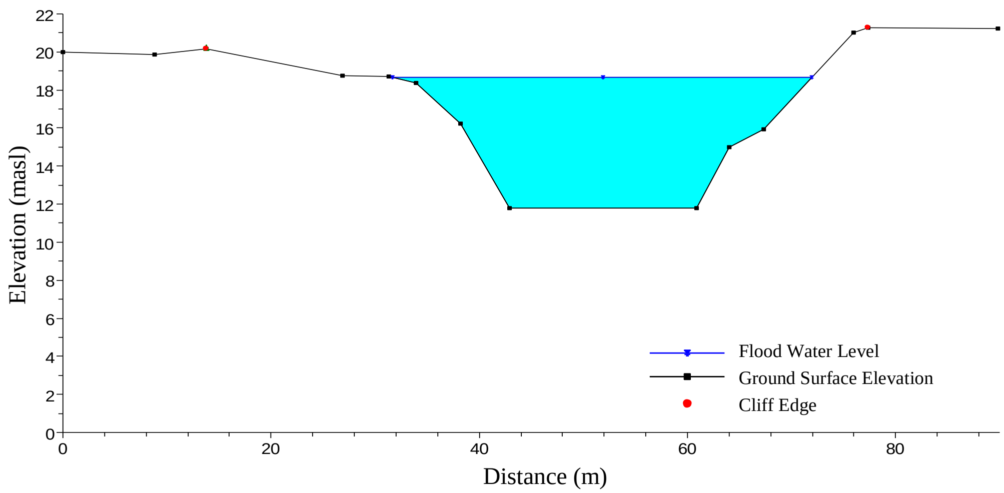

Figure 7. Cross-section of the Bekasi River at station 1100 after normalization

Dredging of the riverbed was undertaken to address the overflow at Station 1100. As depicted Figure 7, notable alterations in the depth and width of the riverbed occurred. These alterations enlarged the cross-sectional river area, accommodating a more significant water discharge. The original river depth value, which stood at 13.86 masl, decreased to 11.8 masl post-dredging. Consequently, after normalization, the flood water level of the Bekasi River at Station 1100 also decreased, reaching 18.69 masl.

Figure 8 shows the right side of the river at station 11500, which is experiencing overflow because the right side of the river does not yet have an embankment, while the left side already has a 4 m high embankment. The cliffs to the left and right of this station have a height of 27.447 masl and 25.492 masl, respectively, while the MAB value at this station is as high as 27.19 masl so that overflow occurs at the right side of the river. On the left bank of this station, an ineffective flow area exists, which indicates that this area is not part of the river. Water will not flow into the area because the water must cross the embankment first.

The overflow at Station 11500 was overcome by building an embankment. Figure 9 shows that on the right side of the river at station 11500, a 4 m high embankment was created. This causes the cross-sectional area of the river to become larger and able to carry more water. The cliff to the right of this station has an elevation increase from 25.492 masl to 27.470 masl, so the flood water level value at station 11500 has decreased to 24.92 masl.

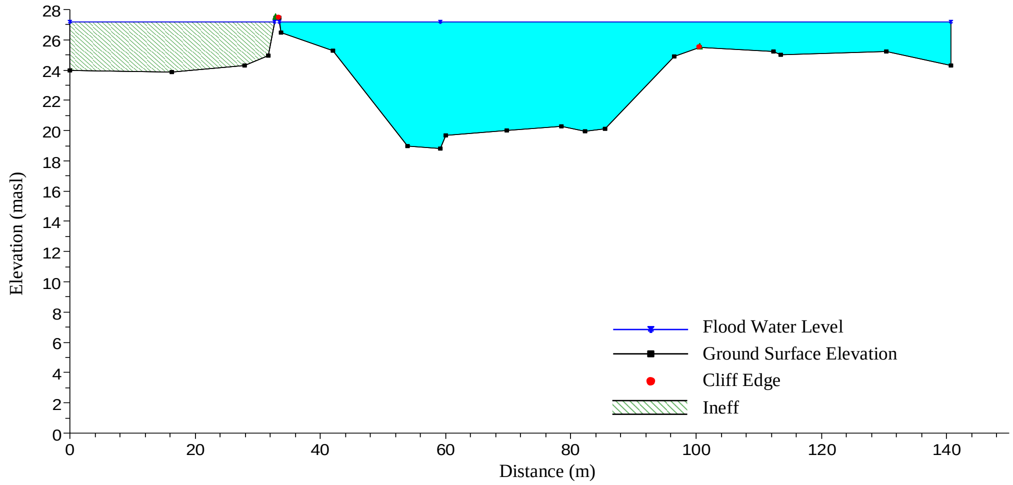

Figure 8. Cross-section of the Bekasi River at station 1500 before normalization

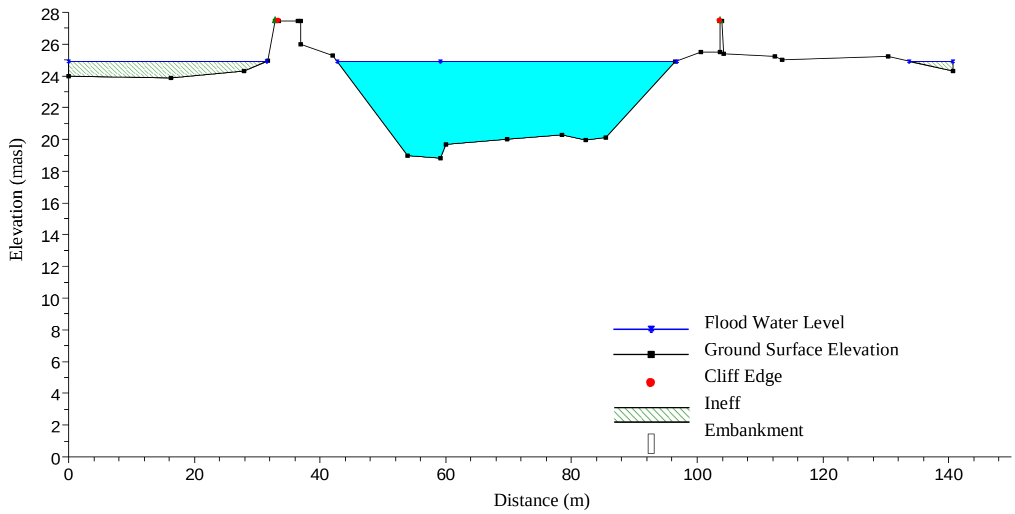

Figure 9. Cross-section of the Bekasi River at station 1500 after normalization

4. CONCLUSIONS

The analysis results using the Nakayasu synthetic unit method indicate that the flood discharge of the Bekasi River for the 25-year return period is 665.81 m^3/s. The analysis reveals that overflow occurred at 102 out of 116 stations along the river. At these 102 overflow stations, the capacities of the river cross-sections range from 453.49 m^3/s to 665.71 m^3/s. After normalization, the capacities of the river cross-sections range from 692.52 m^3/s to 884.91 m^3/s. Consequently, it is necessary to undertake measures to address the overflow. Following river normalization process, which involved new embankment construction and riverbed dredging, the cross-sectional capacity at all Bekasi River stations can accommodate flood discharge without any instances of overflow. Thus, the normalization efforts effectively mitigate flooding in the Bekasi River.

REFERENCES

Andayani, R., & Umari, Z.F. (2022). Debit banjir rancangan DAS Selabung dengan HSS Nakayasu. *Jurnal Deformasi*, ***7***(1), 21–31. https://doi.org/10.31851/deformasi.v7i1.7803

Ardiansyah, M., Suyono, Titisariwati, I., Cahyadi, T.A., & Kresno. (2021). Analisis perbandingan perhitungan curah hujan rencana berdasarkan periode ulang hujan dengan metode Gumbell, metode log Pearson III, metode Iway Kadoya studi kasus tambang andesit. *Jurnal Inovasi Pertambangan dan Lingkungan*, ***1***(2), 11–16. https://doi.org/10.15408/jipl.v1i2.22731

BSN (Badan Standardisasi Nasional). (2016). *SNI 2415 Tentang Tata Cara Perhitungan Debit Banjir Rencana*.

Bagaskara, D.F., Bisri, M., & Andawayanti, U. (2024). Studi evaluasi sumur resapan di Kelurahan Cipinang Besar, Kecamatan Jatinegara, Kota Jakarta Timur. *Jurnal Teknologi dan Rekayasa Sumber Daya Air*, ***04***(01), 78–90. https://doi.org/10.21776/ub.jtresda.2024.004.01.007

Damayanti, A.C., Limantara, L.M., & Haribowo, R. (2022). Analisis debit banjir rancangan dengan metode HSS Nakayasu, HSS ITB-1, dan HSS Limantara pada DAS Manikin di Kabupaten Kupang. *Jurnal Teknologi dan Rekayasa Sumber Daya Air*, ***2***(2), 300–313. https://doi.org/10.21776/ub.jtresda.2022.002.02.25

Djunit, N., & Aziz, M.R.A. (2020). Mitigasi bencana banjir (studi kasus di Kecamatan Rawalumbu Kota Bekasi). *Jurnal Ilmiah Plano Krisna*, ***16***(2), 220–231.

Feyen, J. (1980). *Drainage of Irrigated Land*. Batsford Academic and Educatonial Ltd, Katholieke Universitet Leuven, Center for Irrigation Engineering.

Herison, A., Romdania, Y., Purwadi, O.T., & Effendi, R. (2018). Kajian penggunaan metode empiris dalam menentukan debit banjir rancangan pada perencanaan drainase (review). *Jurnal Aplikasi Teknik Sipil*, ***16***(2), 77–86. https://doi.org/10.12962/j2579-891X.v16i2.3819

Husna, S.L., Safriani, M., & Farizal, T. (2022). Perbandingan analisis debit banjir menggunakan hidrograf satuan sintetis (HSS) Snyder dan Nakayasu pada Sungai Krueng Tripa. *Jurnal Media Teknik Sipil Samudra*, ***3***(2), 8–19. https://doi.org/10.55377/jmtss.v3i2.5695

Kodoatie, R.J., & Sugiyanto. (2002). *Banjir, Beberapa Penyebab dan Metode Pengendaliannya dalam Perspektif Lingkungan*. Pustaka Pelajar.

Krisnayanti, D.S., Bunganaen, W., Hangge, E.E., Munaisyah, F., & Khaerudin, N.D. (2018). Analisis nilai koefisien limpasan permukaan pada embung kecil di Pulau Flores bagian timur. *Jurnal Sumber Daya Air*, ***14***(2), 125–140. https://doi.org/10.32679/jsda.v14i2.325

Limantara, L.M. (2010). *Hidrologi Praktis*. CV Lubuk Agung, Bandung: 336 pp.

Margini, N.F., Nusantara, D.A.D., & Ansori, M.B. (2017). Analisa hidrograf satuan sintetik Nakayasu dan ITB pada sub DAS Konto, Jawa Timur. *Jurnal Hidroteknik*, ***2***(1), 41–45.

Rojali, A., & Elsari, P. (2020). Pemodelan banjir di Perumahan Pondok Gede Permai Bekasi. *Rekayasa Sipil*, ***9***(1), 6. https://doi.org/10.22441/jrs.2020.v09.i1.02

Ruhiat, D. (2022). Implementasi distribusi peluang gumbel untuk analisis data curah hujan rencana. *Teorema: Teori Dan Riset Matematika*, ***7***(1), 213–234. https://doi.org/10.25157/teorema.v7i1.7137

Soemarto, C. (1999). *Hidrologi Teknik*. Erlangga, Jakarta: 313 pp.

Soewarno, S. (1995). *Hidrologi Aplikasi Metode Statistik untuk Analisa Data*. Nova, Bandung: 269 pp.

Suripin, S. (2004). *Sistem Drainase Perkotaan yang Berkelanjutan*. [ISBN: 9797311376] Andi Offset, Yogyakarta: 384 pp

Triatmodjo, B. (2010). *Hidrologi Terapan*. [ISBN: 978-979-8541-40-7]. Beta Offset, Yogyakarta: 358 pp.

Upomo, T.C., & Kusumawardani, R. (2016). Pemilihan distribusi probabilitas pada analisa hujan dengan metode goodnes of fit test. *Jurnal Teknik Sipil Perencenaan*, ***18***(2), 139–148. https://doi.org/10.15294/jtsp.v18i2.7480

Yunida, R., Kumalawati, R., & Arisanty, D. (2017). Dampak bencana banjir terhadap kondisi sosial ekonomi masyarakat di Kecamatan Batu Benawa, Kabupaten Hulu Sungai Tengah, Kalimantan Selatan. *Jurnal Pendidikan Geografi*, ***4***(4), 42–52.