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Evaluation of Rainfall Intensity and the Effectiveness of Green Infrastructure in Urban Water Management: A Case Study of the University of Kadiri

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Abstract: Urban flooding in tropical regions like Indonesia is increasingly critical due to rapid urbanization and climate change, leading to reduced infiltration and overloaded drainage systems. This study analyzes rainfall intensity and drainage capacity at Kediri University using 15 years of daily maximum rainfall data and four statistical distributions Gumbel, Pearson Log III, Normal, and Log-Normal. Goodness-of-fit tests identify Pearson Log III as the best model for design rainfall estimation. Runoff discharge is calculated via the Rational Method and compared to the existing drainage capacity, revealing insufficient infrastructure for rainfall events with return periods over 10 years. To address this, the study evaluates infiltration wells as green infrastructure solutions, demonstrating their effectiveness in reducing runoff and enhancing groundwater recharge. This integrated approach bridges the gap between statistical rainfall modeling and practical drainage performance assessment in localized tropical urban contexts. The results offer actionable insights for sustainable stormwater management and flood mitigation on university campuses and similar settings, contributing both theoretically and practically to urban water management under evolving climatic challenges.

Keywords: Green Water, Rainy, Water Potential

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Introduction

Urban flooding has emerged as a significant challenge in tropical countries such as Indonesia, driven primarily by rapid urbanization and the accelerating impacts of climate change (Susanto, Winarto, et al., 2024). The conversion of permeable natural land into impervious surfaces like buildings and roads drastically reduces rainwater infiltration, resulting in increased surface runoff and overburdened drainage systems. Data from the Meteorological Agency's Stasiun Klimatologi Jawa Timur (BMKG Station No. 96943, latitude -7.90080°, longitude 112.59790°, elevation 590 meters) indicate that daily maximum rainfall has reached up to 145 mm in certain years between 2009 and 2023 (BMKG, 2024)**.** This data can be seen in Figure 1 and Table 1 below.

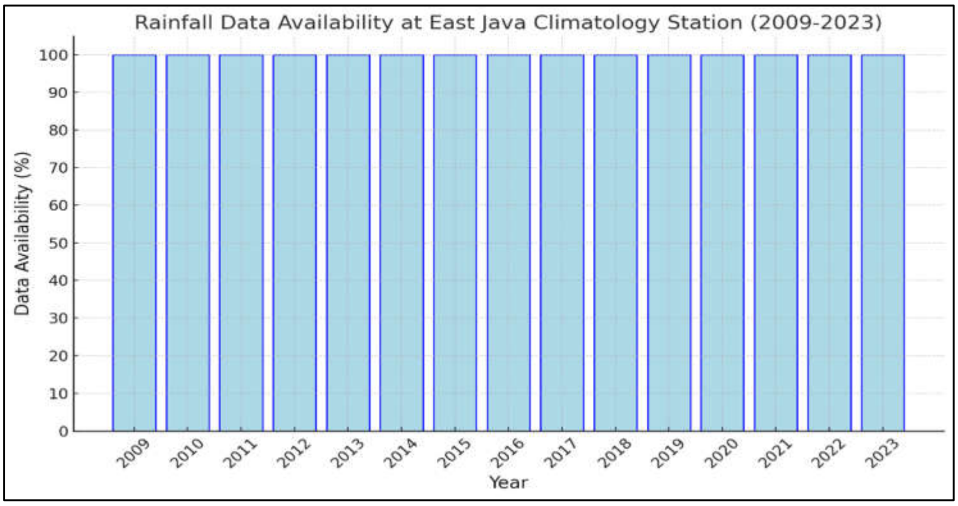

Figure 1 Rainfall Data (BMKG, 2024)

Table 1 Stasiun Data

Station Name	East Java Climatology Station
Station Number	96943
Station Type	UPT (Technical Implementation Unit)
Region	Region III
Province	East Java
City	Malang City
Longitude	112.59790
Latitude	-7.90080
Altitude	590 m
Soil Exposure	52.00000
Land Use	-
Time Zone	UTC +07:00

At Universitas Kadiri, this problem is accentuated by the campus landscape, which is dominated by buildings and paved roads, leading to a high compound runoff coefficient of approximately 0.82. Observations show that the existing drainage channels, with dimensions of 0.5 by 0.6 meters, have a maximum discharge capacity of 0.51 m^3/s, while runoff during heavy rainfall events can reach 0.62 m^3/s, surpassing channel capacity and increasing the risk of flooding (Hu et al., 2021; Jeong et al., 2025; Zuo et al., 2025).

Rainfall frequency analysis and runoff estimation in Indonesia commonly employ Intensity-Duration-Frequency (IDF) curves and the Rational Method, utilizing statistical distributions such as Gumbel, Log Pearson III, Normal, and Log-Normal (Alzahrani et al., 2025; Badhurahman et al., 2021; Kencanawati et al., 2021). However, many studies neglect rigorous validation of these models and fail to consider local hydrological conditions, including soil moisture variability and land heterogeneity, which are critical to accurate runoff predictions (Campos-Aranda, 2023; Farahmand et al., 2025; Faye, 2022; Lukk et al., 2023). Green infrastructure solutions, particularly infiltration wells, have shown potential in reducing surface runoff and enhancing groundwater recharge in tropical urban settings (Ferreira et al., 2024; Moravej et al., 2025; Wayan Mundra & Wedyantadji, 2021). Nonetheless, their integration into practical drainage planning remains limited, especially at the scale of university campuses (Zuo et al., 2025). This study aims to address these gaps by integrating statistical rainfall modeling, runoff estimation, drainage system capacity assessment, and infiltration well design based on national technical standards (SNI 8456:2017). The research applies multiple rainfall distribution models validated by goodness-of-fit tests and assesses drainage performance using field data, offering a comprehensive approach to sustainable urban water management at the campus scale.

Literatur Review

Rainfall Statistical Analysis

Statistical modeling of rainfall extremes is fundamental in urban hydrology to estimate design rainfall for infrastructure planning. Commonly used probability distributions include Gumbel, Log Pearson III, Normal, and Log-Normal (Alzahrani et al., 2025; Koutrouvelis & Canavos, 1999). However, studies often fail to critically evaluate the goodness-of-fit or discuss the implications of distribution choice on flood prediction accuracy (Campos-Aranda, 2023). Recent research emphasizes the importance of validating distribution models using multiple tests such as Kolmogorov–Smirnov and Chi-square to ensure robust design rainfall estimation (Moccia et al., 2021). Additionally, the sensitivity of extreme rainfall estimates to skewness and sample size highlights the need for careful data analysis, especially in tropical climates where rainfall patterns differ from temperate regions (Montes-Pajuelo et al., 2024).

Urban Runoff and Drainage Capacity

The Rational Method remains widely used for quick estimation of peak runoff in urban catchments, relying on runoff coefficients that depend on land use and soil characteristics (Kencanawati et al., 2021). Despite its simplicity, the method assumes uniform rainfall distribution and catchment characteristics, which can limit accuracy if not calibrated with local data (Badhurahman et al., 2021). Field measurements and hydraulic modeling tools such as SWMM have been employed to improve understanding of drainage system performance and flood risks (Asferizal et al., 2024; Ferreira et al., 2024). However, there is a scarcity of studies that integrate statistical rainfall modeling with real drainage system evaluation in tropical campus environments.

Green Infrastructure and Stormwater Management

Green infrastructure, including infiltration wells and vertical green systems, has gained attention as a sustainable approach to mitigate urban runoff and enhance groundwater recharge (Moravej et al., 2025; Wayan Mundra & Wedyantadji, 2021). These nature-based solutions contribute to flood risk reduction and improve environmental quality by promoting infiltration and evapotranspiration (Ferreira et al., 2024). Nevertheless, their practical implementation in tropical urban contexts, especially in localized settings like university campuses, remains underexplored (Zuo et al., 2025). Recent studies suggest that integrating green water management into urban planning requires site-specific hydraulic and hydrological assessments to maximize effectiveness (Lukk et al., 2023).

Research Method

This study began with a comprehensive literature review to gather relevant data related to rainfall characteristics and existing water management practices in the study area. Daily rainfall records spanning the last 15 years were obtained from the official database of the Indonesian Meteorology, Climatology, and Geophysics Agency (BMKG). From these records, the maximum daily rainfall values for each year were extracted to build a dataset for further statistical analysis. These data were analyzed to calculate key statistical parameters such as mean, standard deviation, skewness, and kurtosis, which are essential to understand the rainfall distribution characteristics in the local tropical climate context.

Land use data were collected using high-resolution Google Earth imagery supplemented with field surveys to validate and classify land cover into distinct categories such as buildings, green areas (parks), roads, and vacant lands. Each land use category was assigned an appropriate runoff coefficient value based on surface permeability and water absorption properties from established hydrological literature. These coefficients were then integrated with their respective land areas to compute a compound runoff coefficient, representing the overall runoff potential for the study area.

The core of the hydrological analysis involved modeling the frequency distribution of annual maximum rainfall using four widely recognized probability distributions: Gumbel, Pearson Type III Logarithmic, Normal, and Log-Normal. Each distribution was fitted to the data by estimating their statistical parameters, followed by rigorous goodness-of-fit tests using the Kolmogorov-Smirnov and Chi-Square methods at significance levels of 1% and 5%. This step ensured that the selected distribution accurately represents the empirical rainfall data, which is critical for reliable flood risk estimation and infrastructure design. Following the determination of the best-fitting rainfall distribution, the design rainfall for various return periods was computed. These design values were then used to calculate the planned peak flood discharge employing the Rational Method, a conventional yet practical hydrological approach suitable for urban catchments. To assess the adequacy of existing drainage infrastructure, field measurements of the drainage channel dimensions and conditions were conducted. The channel's capacity to convey the calculated peak discharge was analyzed to identify potential bottlenecks or flood risk areas.

In cases where the existing drainage channels were found insufficient to accommodate peak flows, the study evaluated green infrastructure solutions, specifically infiltration wells, as a complementary measure to reduce surface runoff and enhance groundwater recharge. The design and sizing of infiltration wells followed national technical standards (SNI 8453:2017), considering soil permeability and rainfall intensity. This combined quantitative and field-verified methodology aligns with reviewer recommendations by integrating statistical rigor, empirical validation, and practical flood mitigation strategies, thereby enhancing the study's applicability and reliability for local urban water management (Figure 2).

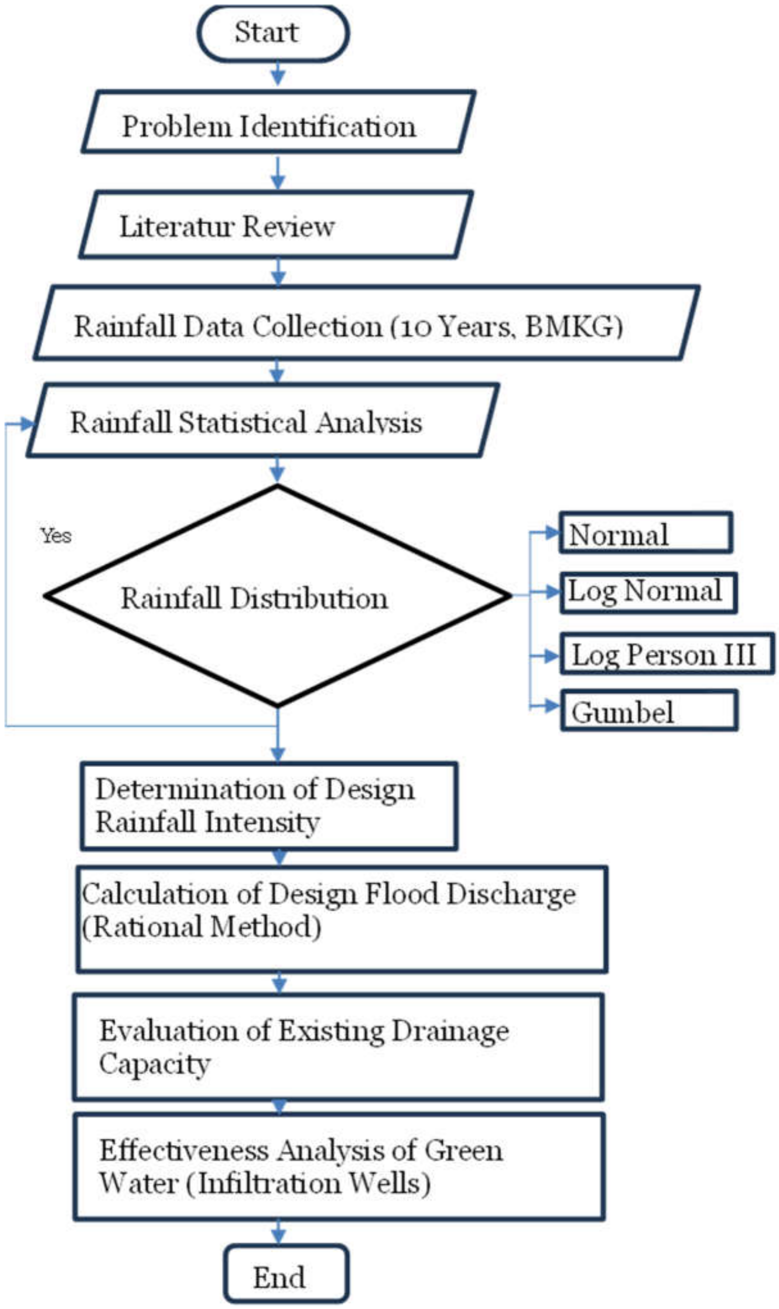

Figure 2 Flowchart Research

This study uses a quantitative-descriptive case study design to analyze rainfall intensity and drainage performance at Kadiri University. The research instruments include secondary rainfall data from the East Java DPU, topographic maps to identify watershed characteristics, and field observation sheets to assess drainage channel conditions. Rainfall intensity is modeled using Gumbel, Pearson Log III, Normal, and Log-Normal distributions. The Modified Rational Method is applied to estimate peak discharge, which is then compared to actual channel capacity. These analyses support recommendations for green-water management strategies such as infiltration wells.

Result and Discussion

Research Location and Land Use Analysis

The study was conducted at Kadiri University, covering an area of 18,710.89 m². Land use classification was conducted based on Google Earth imagery in 2024, revealing that the majority of the area is occupied by buildings (9,825.76 m²), followed by vacant land, roads, and gardens (Table 2). These variations in land use contribute to different surface runoff coefficients due to their diverse water absorption capacities.

Table 2 Calculation of runoff coefficient

Land Use	Area (m²)	Runoff Coefficient (C)
Building	9,825.76	0.95
Garden	1,091.31	0.45
Road	2,662.44	0.70
Vacant Land	5,131.38	0.70
Total	18,710.89	

The compound runoff coefficient, calculated as 0.82, reflects the high imperviousness of the campus area, indicating that most rainfall converts to surface runoff (Hu et al., 2021). This coefficient plays a pivotal role in estimating flood discharge and designing drainage systems.

Rainfall Data and Frequency Analysis

Annual maximum daily rainfall data over the past 15 years were sourced from BMKG (Figure 3). This dataset serves as the basis for frequency distribution modeling and flood risk assessment. The data reveal variability and occasional extreme rainfall events, with the highest recorded annual daily rainfall of 145 mm in 2021. The maximum annual rainfall (RMAX) data from 2009 to 2023 shows varying rainfall levels over the years. In 2009, the rainfall was recorded at 82 mm, and it decreased slightly to 68 mm in 2010. The rainfall increased in 2011 to 78 mm, reaching 98 mm in both 2012 and 2013. The data from 2014 to 2016 remained relatively high, with rainfall values of 96.1 mm, 91.6 mm, and 97.1 mm, respectively. In 2017, the rainfall dropped to 87 mm but increased again to 107.4 mm in 2018. For the following years, the rainfall remained between 84.6 mm and 96.7 mm, except for 2021, which experienced a peak of 145 mm. The data showed a slight decrease in 2022 to 95.7 mm, and in 2023, the rainfall dropped further to 70.9 mm. This fluctuation in rainfall reflects the variability and changing patterns in rainfall intensity over time (Figure 3).

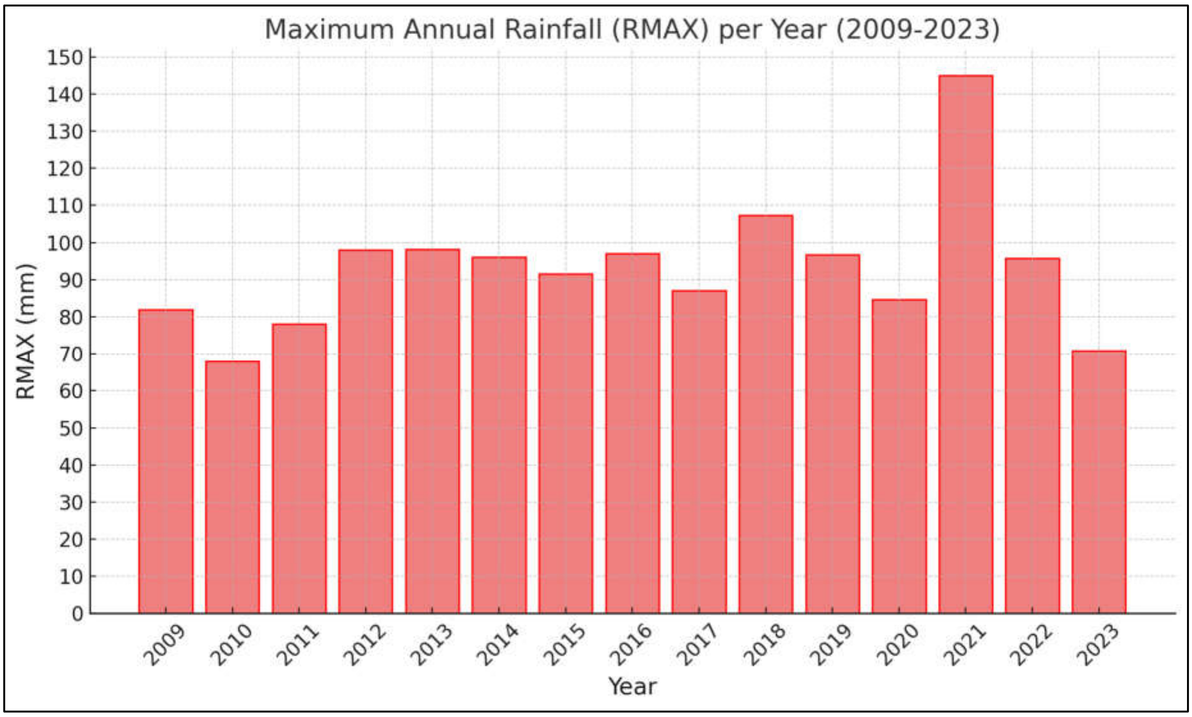

Figure 3 Annual maximum rainfall data (BMKG, 2024)

As an introduction to the statistical analysis, the observed variation in annual maximum rainfall data highlights the importance of selecting an appropriate probability distribution to accurately estimate design rainfall. The recorded data serves as a foundation for hydrological modeling, which requires the application of statistical methods to gain meaningful insights. In this study, four types of statistical distributions Gumbel, Pearson Log III, Normal, and Log-Normal, were applied to the dataset. Goodness-of-fit tests, namely Smirnov-Kolmogorov and Chi-Square, indicated that all four distributions are statistically valid.

Gumbel Distribution

The calculation of the Gumbel distribution is carried out by first looking for several statistical parameters as follows:

Table 3 Gumbel statistical parameters

Average (x)	93,09
Standard deviation (Sd)	18,08
Lots of data (n)	15
Cs	1,55
Ck	7,72
Yn	0,5128
Sn	1,0206

From table 4, the Average and Standard Deviation values are taken based on the data in table 3 with an average value of 93.09 and a standard deviation of 18.5. The skewnes coefficient (Cs) is sought with the following equation (Alzahrani et al., 2025; Badhurahman et al., 2021):

$$Cs = \frac{n\sum(x-\bar{x})^3}{(n-1)(n-2)S^3} \tag{1}$$

Where is the sum of the three powers of the data subtraction with the average of the data. (n) is the amount of data and (S) is the deviation so that the $\sum(x-\bar{x})^3$ skewnes coefficient (Cs) value of 1.55 is obtained (Alzahrani et al., 2025; Campos-Aranda, 2023). The kurtosis coefficient (Ck) is calculated by the following equation (Badhurahman et al., 2021).

$$Ck = \frac{n^2\sum(x-\bar{x})^4}{(n-1)(n-2)(n-3)S^4} \tag{2}$$

$\sum(x-\bar{x})^4$ Where is the sum of the four powers of the data subtraction with the average of the data. From the calculation of the equation, a value (Ck) of 7.72 was obtained. The values of the Yn and Sn coefficients were searched based on the amount of data matched on the Gumbel distribution table so that the Yn value was 0.5128 and the Sn value was 1.0206. A recap of the rainfall calculation of the Gumbel distribution design with a certain repetition time (Tr) can be seen in table 4 below.

Table 4 Gumbel Distribution

Repeat Time (Tr)	YT	K	Design(mm)
2	0,37	-0,14	90,49
5	1,50	0,97	110,58
10	2,25	1,70	123,88
25	3,20	2,63	140,68
50	3,90	3,32	153,14
100	4,60	4,00	165,51
200	5,30	4,69	177,84
1000	6,91	6,27	206,39

The value of the YT variable in table 5 above is calculated using the equation (Alzahrani et al., 2025; Badhurahman et al., 2021):

$$YT = -ln\left[ln\left(\frac{Tr}{Tr-1}\right)\right] \tag{3}$$

Where the variable Tr is a certain repetition period. The value of the variable K in table 5 is calculated by the equation (Badhurahman et al., 2021; Campos-Aranda, 2023):

$$K = \frac{YT - Yn}{Sn} \tag{4}$$

Where the values of the variables Yn and Sn are taken from table 4. The planned rainfall value (R plan) is calculated based on the variables YT and K with the following equation (Alzahrani et al., 2025; Badhurahman et al., 2021):

$$Rrancangan = \bar{x} + K.S \tag{5}$$

$\bar{x}$ Where is the average of the data and S is the standard value of the deviation

Pearson III Log Distribution

The calculation of the Pearson III Log distribution is done by first changing the data value in table 5 with its logarithm (Alzahrani et al., 2025; Koutrouvelis & Canavos, 1999).

Table 5 Logarithmic results of annual maximum rainfall data

Year	RMAX (mm)	Log R max
2009	82	1,913813852
2010	68	1,832508913
2011	78	1,892094603
2012	98	1,991226076
2013	98,2	1,992111488
2014	96,1	1,982723388
2015	91,6	1,961895474
2016	97,1	1,98721923
2017	87	1,939519253
2018	107,4	2,031004281
2019	96,7	1,985426474
2020	84,6	1,927370363
2021	145	2,161368002
2022	95,7	1,980911938
2023	70,9	1,850646235

The R max Log value is obtained from the rmax value in each year. From the data in table 6 above, the value of several parameters is searched for the calculation of the Pearson III log distribution with the following results.

Table 6 Statistical parameters of Pearson III Log

Average Log R	1,961989305
Standard deviation (S log R)	0,078523364
Lots of data (n)	15
Cs	0,731

The calculation of the Pearson III Log distribution is done by first changing the data value in table 7 with its logarithm (Alzahrani et al., 2025; Koutrouvelis & Canavos, 1999)

$$Cs = \frac{n\sum(Log\ x - Log\bar{x})^3}{(n-1)(n-2)S^3} \tag{6}$$

$\sum(Log\ x - Log\bar{x})^3$Where is the sum of the power of three of the subtraction of the logarithmic data value with the average of the logarithmic data. From the above equation, the value of the skewnes coefficient (Cs) is 0.731. A recap of the planned rainfall calculation with the distribution of Pearson Log III with specific repetitions is presented in table 7 below.

Table 7 Pearson III Log Distribution

Birthday [tr] (year)	K	R log	R design(mm)
2	-0,12082	1,952502	89,64005
5	0,78694	2,023782	105,6288
10	1,33412	2,066749	116,6135
25	1,97494	2,117068	130,9388
50	2,42117	2,152108	141,941
100	2,8446	2,185357	153,2346
200	3,25069	2,217244	164,909
1000	4,14496	2,287466	193,8499

The value of the K coefficient in table 7 above is searched by matching the values of the Skewnes coefficient (cs) and the specific repetition period (Tr) in the Pearson III Log distribution coefficient table. The logarithmic value of the planned rainfall (R log) for the Pearson III Log distribution is calculated by taking into account the variable K with the following equation (Koutrouvelis & Canavos, 1999; Montes-Pajuelo et al., 2024).

$$R\ log = \overline{x\ log} + K.Slog \tag{7}$$

$\overline{x\ log}$Where is the average of the data R that has been logarithmic first. The value of the planned rainfall (R log) is obtained by exponential the value of (R log).

Normal Distribution

The calculation of the Normal distribution is carried out by first looking for several statistical parameters as follows, see Table 8 below.

Table 8 Normal Distribution Statistical Parameters

Average R	Standard deviation (Sd)	Lots of data (n)
93,09	18,08	15

The average R-value is the average value of the maximum rainfall data in table 3. Likewise, the standard deviation value (Sd) is the standard deviation value for the data in table 3 with the number of data (n) as many as 15 data. A recap of the rainfall calculation of the Normal distribution plan for a given repetition (Tr) can be seen in table 9 below.

Table 9 Normal Distribution

Birthday [tr] (year)	K	Sd.K	Design(mm)
2	0,00	0,00	93,09
5	0,84	15,19	108,28
10	1,28	23,15	116,23
25	1,75	31,65	124,73
50	2,05	37,07	130,16
100	2,33	42,14	135,22
200	2,58	46,66	139,74
1000	3,09	55,88	148,97

The value of the variable K in table 9 above is obtained from the table of Gauss reduction variables based on a specific repetition (Tr) value. The value of the variable (Sd. K) is the multiplication between the Gauss reduction coefficient (K) and the standard deviation (Alrweili, 2024; Alzahrani et al., 2025). The planned rainfall value (R plan) is calculated using the following equation:

$$Rrancangan = \bar{x} + Sd.K$$

Where is the average of the data $\bar{x}$

Normal Log Distribution

The calculation of the distribution of the Normal Log is carried out by first cataloguing the data as in table 6 so that several statistical parameters are obtained as follows, see Table 10 below.

Table 10 Normal Log Distribution Statistical Parameters

Average Log R	Standard deviation (Sd Log)	Lots of data (n)
1,962	0.079	15

The average value of Log R is the average value of the R max log data in table 6. Similarly, for the value (S log R) is the standard value of Deviation for log data R max in table 6 with a number of data (n) as many as 15 data. mA recap of the rainfall calculation of the Normal Log distribution plan for a given repetition (Tr) can be seen in table 11 below.

Table 11 Normal Log Distribution

Birthday [tr] (year)	K	R Log	R design(mm)
2	0	1,96	91,62
5	0,84	2,03	106,65
10	1,28	2,06	115,48
25	1,75	2,10	125,72
50	2,05	2,12	132,73
100	2,33	2,14	139,62
200	2,58	2,16	146,08
1000	3,09	2,20	160,19

The value of the variable K in table 9 above is obtained from the table of Gauss reduction variables based on a specific repetition (Tr) value. The value of the variable (Sd.K) is the multiplication between the Gauss reduction coefficient (K) and the standard deviation (Alrweili, 2024; Alzahrani et al., 2025).

Frequency Distribution Selection

From the calculation of the distribution with the 4 (four) frequency distribution methods above, then a distribution test was carried out with the Smirnov Kolomogorov test method and the Chi-Square Test with a confidence degree of 1% and 5%. Critical values for the Smirnov Kolomogorof and Chi Square tests are based on tables 12 and 13 below (Alzahrani et al., 2025; Bina & Moradinia, 2024).

Table 12 Critical value of Smirnov kolomogorov

Sample size (n)	Degree of trust (a)			
	0,2	0,1	**0,05**	**0,01**
5	0,45	0,51	0,56	0,67
10	0,32	0,37	0,41	0,49
15	0,27	0,3	0,34	0,4

Rainfall data up to the last 15 years from table 12 above was taken as a critical value for the Smirnov kolomogorof test with a confidence degree of 1% of 0.4 and 5% of 0.34 (Alzahrani et al., 2025; Bina & Moradinia, 2024).

Table 13 Chi Square's critical value

Degree of Freedom (dk)	Degree of trust (a)			
	0,05	0,025	0,01	0,005
1	3,841	5,024	0,635	7,879
2	5,991	0,738	9,210	10,597
3	7,815	9,348	11,345	12,838

As for the Chi Square test, from table 13 above, a critical value was taken for the degree of freedom 2 with a confidence degree of 1% of 9.210 and 5% of 5.991 (Shen et al., 2022). The critical value of each of the above distribution tests is then compared with the calculation results of the Smirnov kolomogorof test and the Chi-square test with the following results:

Table 14 Recap Test spread

Plan R (mm)				
Birthday [tr] (year)	**D. Gumbel**	**D. Log Pearson III**	**D. Normal**	**D. Normal Logs**
2	90,49	89,64	93,09	91,62
5	110,58	105,63	108,28	106,65
10	123,88	116,61	116,23	115,48
25	140,68	130,94	124,73	125,72
50	153,14	141,94	130,16	132,73
100	165,51	153,23	135,22	139,62
200	177,84	164,91	139,74	146,08
1000	206,39	193,85	148,97	160,19
SMIRNOV-KOLMOGOROF TEST				
Δcritical 1%	0,34	0,34	0,34	0,34
Δcritical 5%	0,40	0,40	0,40	0,40
Δcount	0,096	0,14	0,200	0,163
Conclusion (1%)	Accepted	Accepted	Accepted	Accepted
Conclusion (5%)	Accepted	Accepted	Accepted	Accepted
CHI-SQUARE TEST				
X2 (1%)	5,991	5,991	5,991	5,991
X2 (5%)	9,21	9,21	9,21	9,21
X2Count	4,666667	4	3,333333	2
Conclusion (1%)	Accepted	Accepted	Accepted	Accepted
Conclusion (5%)	Accepted	Accepted	Accepted	Accepted

The smirnov test of the Gumbel distribution column obtained a critical value of 0.096, the Pearson log distribution of 0.14, the normal distribution of 0.2 and the normal log distribution of 0.165. From the test values of the above distribution to everything below from the critical value which means that all distributions are acceptable. The Chi square test of the Gumbel distribution obtained a critical value of 4.6, the Pearson log distribution of 04, the normal distribution of 3.33 and the normal log distribution of 2. For the Chi square test to everything is also below the critical value which means that all distributions are acceptable. Distribution testing with statistical parameters is performed to select which distribution is acceptable if the smirnov test and the Chi square test to all are accepted (Moccia et al., 2021).

Table 15 Test statistical parameters

Distribution Types	Condition	Result	Ket.
Usual	Cs≈0	1,55	not
	Ck = 3	7,72	not
Gumbel	CS≤ 1.1396	1,55	not
	CK≤ 5.4002	7,72	not
Log Pearson III	Cs ≠ 0	0,73	Ok
Normal Logs	Cs≈Cv 3+ 3Cv	0,73	not
	Ck = 5,383	5,61	not

The Skewnes coefficient (Cs) is calculated by equation 1 with the data in table 3 obtained a value of 1.55. The curtosis coefficient (Ck) was calculated with equation 2 with the data in table 3 obtained a value of 7.72. From table 15 above, it is found that the distribution of Pearson III logs can be accepted so that the proposed rainfall value in this distribution used in the planning is 105.6288 mm at the 5-year replay period (Table 8).

Discharge Flood Plan

From the results of the rainfall calculation in the design above, it is used to calculate the intensity of rainfall with the Mononobe equation as follows (Rakhmawati, 2024):

$$I = \frac{R_{24}}{24}\left(\frac{24}{t}\right)^{\frac{2}{3}} \quad (8)$$

the value of R24 is equal to the design rainfall value of 105.6288 mm. The concentration time value (t) is calculated by adding the inlet time (t_o) and the channel flow time (t_d). For the inlet time (t_o) is calculated by the kirpich formula approach as follows (Rakhmawati, 2024).

$$t_o = 0{,}0195\left(\frac{L_o}{\sqrt{S_o}}\right)^{0{,}77} \quad (9)$$

to is the farthest flow distance above the ground to the main channel. After being calculated at the research location, a value of 219.4 m was obtained. While so is the slope, after calculating it obtained a value of 0.021. So that after being calculated with equation 7, it gets a to value of 0.09 hours. (t_d) is the time when water flows from upstream to downstream on a channel. The value of td was measured in the field and a value of 0.02 hours was obtained. Concentration time (t) is the sum of the values to, and td and a value of 0.11 hours is found so that the value of rain intensity can be calculated with the above parameters using the Mononobe formula as follows.

$I = \frac{105,6288}{24}\left(\frac{24}{0,11}\right)^{\frac{2}{3}}$ = 155.38 mm/h

The rainfall intensity value obtained from the above calculation is used to calculate the design flood discharge (Qr) by rational modification method with the following equation (Kencanawati et al., 2021):

$$Qr = 0,278. C. Cs. I. A \tag{10}$$

Where C is the flow coefficient with a value of 0.82 (table 2) and Cs is the deviation coefficient calculated by the following equation (Al-Amri et al., 2023):

$$Cs = \frac{2\,Tc}{2\,Tc + Td} \tag{11}$$

$Cs = \frac{2\,x\,0,11}{2\,x\,0,11+0,02}$ = 0.91

I am the intensity of rain calculated with equation 6 and A is the area observed to be 0.019 km², from the above parameters, the value of the design flood discharge is calculated with an equation of 10 so that a value of 0.62 m3/second is obtained. This value will be used as a reference in Green Water management planning.

Existing Channel Capacity

Observations in the field were carried out to determine the existing dimensions of the channel. From the results of observations in the field, the research location has a drainage channel with a square cross-section measuring 0.5 x 0.6 m along 198 m with a channel slope (i) of 0.0202, see Figure 4 below.

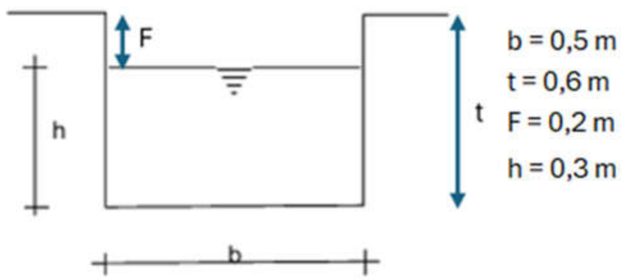

Figure 4 Dimensions of existing channels

From the data on the dimensions of the existing channel overcome, the measurement of the channel discharge capacity (Qout) can be carried out with the following equation (Asferizal et al., 2024):

$$Qout = V. Ac \tag{12}$$

Where V is the flow rate and Ac is the wet cross-sectional area. The wet cross-section area is a cross-sectional area that is submerged in water. The guard height is taken as 0.2 m so that the depth of the existing channel (h) becomes 0.4 m. The wet cross-section area is calculated by multiplying the depth of the channel (h) by the width of the channel (b) until it is found that the value Ac = 0.2 m2. The flow rate (V) is calculated by the equation (Asferizal et al., 2024):

$$V = \frac{1}{n} R^{\frac{2}{3}} i^{\frac{1}{2}} \tag{13}$$

Where the variable n is the manning coefficient, a value of 0.016 is taken for concrete channels with good condition. The variable R is the hydraulic radius of the channel calculated by the equation:

$$R = \frac{A_c}{P} \tag{14}$$

Where P is the wet circumference of the channel with a value of 1.3 m so that the value of the hydraulic radius (R) can be calculated with equation 14 with a result of 0.15385 m. By entering the value of the hydraulic radius variable (R) and the channel slope variable (i) in equation 13, it was found that the flow velocity value was 2.55 m/second.The channel discharge is calculated with equation 12 by entering the flow velocity value (V) with the wet cross-sectional area of 0.51 m3/second.From the calculation above, the maximum channel bit of 0.51 m3/second is not able to accommodate the planned discharge of 0.62 m3/second. Therefore, Green Water management is carried out with the addition of rainwater infiltration wells to accommodate rainwater runoff from buildings.

Infiltration Wells

The calculation of rainwater infiltration wells is carried out according to the SNI-2017 standard regarding rainwater infiltration wells and ditches with the following equations (Wayan Mundra & Wedyantadji, 2021):

$$H = \frac{Q}{5.\pi.r.K} \tag{15}$$

Where H is the depth of the infiltration well required in units of meters, Q is the discharge of the flood contribution with units m3/hour calculated by the equation (Kamalzare et al., 2024; Wayan Mundra & Wedyantadji, 2021):

$$Q = C.I.\,A \tag{16}$$

Q = 0.95 x 0.0231 x 9825.7

Q = 215.34 m3/h

In equation 16, the value of C is taken 0.95 according to the SNI-2017 guidelines. The value I is calculated with equation 8 with the approach t = 2 hours so that a value of 0.0231 m/h is obtained. For area A is the total area of the entire roof of the building. In equation 15, r is the diameter of the infiltration well taken as 1 m. K is the soil permeability coefficient with a value of 0.48 assuming silt soil with moderate permeability properties. From the above parameters, the value of the depth of the infiltration well required with equation 15 was calculated to be 28.57 m. Assuming the depth of the infiltration well is 2 meters. So, 15 units of infiltration wells are needed.

By diverting rainwater runoff from each building, it will reduce the discharge of water runoff flowing into the main channel assuming rainwater runoff from the roof of the building as a whole can be accommodated by infiltration wells. Thus, the area of rainwater runoff accommodated by the main channel will be reduced. Runoff discharge remeasurement was carried out according to equation 10 with the area area approach minus the building area, and the runoff plan emission value of 0.252 m3/second was obtained. With this value, the existing channel is still able to accommodate runoff discharge with the addition of 15 infiltration wells.

Discussion

The study addresses critical gaps identified by previous research and reviewers by rigorously integrating statistical rainfall modeling, hydrological analysis, and practical infrastructure assessment at a local scale. The application of multiple rainfall frequency distributions, supported by goodness-of-fit tests, ensures scientifically valid design rainfall estimations, directly answering Reviewer A's demand for data-backed approaches over generic statements about urbanization and climate effects. The predominance of impervious surfaces on the campus, reflected by the high compound runoff coefficient, underscores the intensified runoff risks typical of rapidly urbanizing tropical regions (Hu et al., 2021; Susanto, Ciptadi, et al., 2024; Susanto, Winarto, et al., 2024). This confirms that land use patterns must be explicitly included in hydrological modeling to prevent underestimation of flood hazards. The finding that existing drainage infrastructure is insufficient to manage design runoff highlights the vulnerability of current urban drainage systems under extreme rainfall intensified by climate change, resonating with observations from Barańczuk et al. (2023) and Jeong et al. (2025). This reinforces the urgency for integrating green infrastructure solutions, which provide both runoff attenuation and groundwater recharge benefits (Ferreira et al., 2024; Moravej et al., 2025).

Infiltration wells demonstrate practical potential in localized urban settings like university campuses, corroborating previous studies emphasizing their role in sustainable stormwater management (Wayan Mundra & Wedyantadji, 2021; Zuo et al., 2025). Their deployment reduces load on conventional drainage infrastructure, mitigating flood risk and contributing to blue-green urban resilience (Ferreira et al., 2024). Despite robust findings, the study recognizes limitations including the reliance on 15 years of historical rainfall data, which may not capture future climate variability comprehensively (Farahmand et al., 2025). Additionally, while the Rational Method provides efficient peak runoff estimation, future research integrating dynamic hydraulic models such as SWMM could provide detailed insights into system performance under variable conditions (Ferreira et al., 2024). In conclusion, this research offers a validated, data-driven framework for urban water management in tropical campuses, combining empirical rainfall analysis with green infrastructure design to support sustainable, resilient urban environments.

Conclusions

This study successfully analyzed rainfall intensity and runoff potential at Kadiri University by applying four statistical distribution models—Gumbel, Pearson Log III, Normal, and Log-Normal and validating them through goodness-of-fit tests. The Pearson Log III distribution was identified as the most appropriate for estimating design rainfall in this tropical context. Runoff calculations using the Rational Method revealed that the current drainage infrastructure is insufficient to handle runoff during rainfall events with return periods greater than 10 years, posing flood risks. The integration of green infrastructure, specifically infiltration wells designed according to national standards, effectively reduces runoff volumes and enables the existing drainage system to operate within capacity limits. These findings demonstrate the critical need for combining statistical rainfall analysis with practical drainage assessments and green infrastructure solutions in urban water management, especially in tropical rapidly urbanizing environments like university campuses. The study contributes a comprehensive, evidence-based approach that can guide planners and campus managers in designing sustainable stormwater systems, mitigating flood risks, and enhancing groundwater recharge. Future work should extend monitoring periods and incorporate dynamic hydraulic modeling for deeper insights into drainage system performance under changing climate conditions.

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