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Performance Evaluation for Surface Run off to Drainage System Normalization at Campus in Jakarta, Indonesia

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ABSTRACT

Drainage is an infrastructure that distributes rainwater from one location to another. On the campus of the Faculty of Engineering at Pancasila University, there are several points of water inundation. Several inundation points were identified based on direct monitoring and measurements at the study site. This study aimed to evaluate the effectiveness of an existing drainage system on campus. The design flood discharge was calculated using a rational method with a five-year return period. From the calculation of hydrology, hydraulics, and comparative analysis of the existing dimensions with the design flood discharge, it was determined that the eight existing channels were unable to accommodate the design flood discharge. Therefore, there are two alternatives for reducing run off discharge. The first is to redesign the drainage channel to improve its size of the drainage channel with a shape that follows the existing shape. The modification involved adjusting the channel height to meet the specified flood discharge requirements. The second alternative was the planning of the infiltration wells. These dimensions are based on the design run off flood discharge, and from the calculation results, the dimensions of the infiltration wells are 1.5 meters in diameter and 2.5 meters high, and four infiltration wells are needed is 4 infiltration wells to reduce the discharge of 1,638 m^3/second.

Introduction

Water is a crucial natural resource for humans and other living organisms. It serves as a transportation medium, energy source, and source of raw water for domestic and industrial purposes [1]. Drainage channels collect rainwater and divert it to holding ponds or rivers [2]. A Watershed is defined as an area that directs water into rivers in the form of groundwater, surface water, or gravity-induced drainage [3,4]. Flood disasters are among the most severe calamities worldwide, given their destructive potential. In addition to physical and financial losses, floods can lead to loss of human life. In Indonesia, a tropical country with distinct rainy and dry seasons, flooding occurs nearly every year in several major cities purposes [5,6]. Addressing these flood issues requires well-planned drainage systems, such as menyeluruh [7–9]. The design of the flood discharge was computed in the hydrological analysis. Design flood discharge represents the maximum flood discharge from a river or canal based on a specific return period [10,11]. This forms the foundation for hydraulic structure planning, ensuring that these structures can accommodate the expected flood volumes during a designated return period [12–14].

Drainage channels within the campus environment are crucial for resident comfort. In the modern era, the increased coverage of asphalt and concrete has led to increased run off and inundation issues. Pancasila University, Jakarta, is no exception, facing inundation challenges in various areas, including the Faculty of Economics parking lot, Anex Park, and Faculty of Engineering parking area. Inadequate drainage channel conditions contribute to inundation, thereby reducing the capacity of the parking area.

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This research focused on evaluating the drainage system within the Pancasila University campus environment, providing a more context-specific and practical approach than general drainage studies [15]. This study explores alternative solutions for mitigating flood discharge, such as redesigning drainage or planning infiltration wells dimensioned according to flood discharges that cannot be managed by existing drainage systems. It also identified specific flood points within the campus, offering a more detailed approach to issues at this location, which may not have been extensively studied. Flood problems, especially in public facilities such as campuses, are closely related to the efficiency of the existing drainage infrastructure, necessitating an evaluation of their effectiveness by assessing the existing channels based on planned return period flood discharges.

Building on these identified issues, this study's primary objective is to comprehensively evaluate the existing drainage channel system within the Faculty of Engineering at Pancasila University. This examination encompasses an in-depth analysis of the system's efficiency, functionality, and overall performance, with the ultimate goal of identifying potential areas for improvement and proposing viable solutions. By addressing these concerns, this study aims to contribute valuable insights that can enhance the effectiveness and sustainability of drainage infrastructure, ensuring a conducive environment for academic activities within the Faculty of Engineering.

Material and Methods

Study Area

The research location is at the Faculty of Engineering, Pancasila University, in the South Jakarta area, DKI Jakarta area (Figure 1), the study location was taken in a campus environment because campus drainage evaluation is crucial for comfort, safety, and environmental conservation. This ensures efficient infrastructure, resilience to climate change, resource management, and disaster prevention.

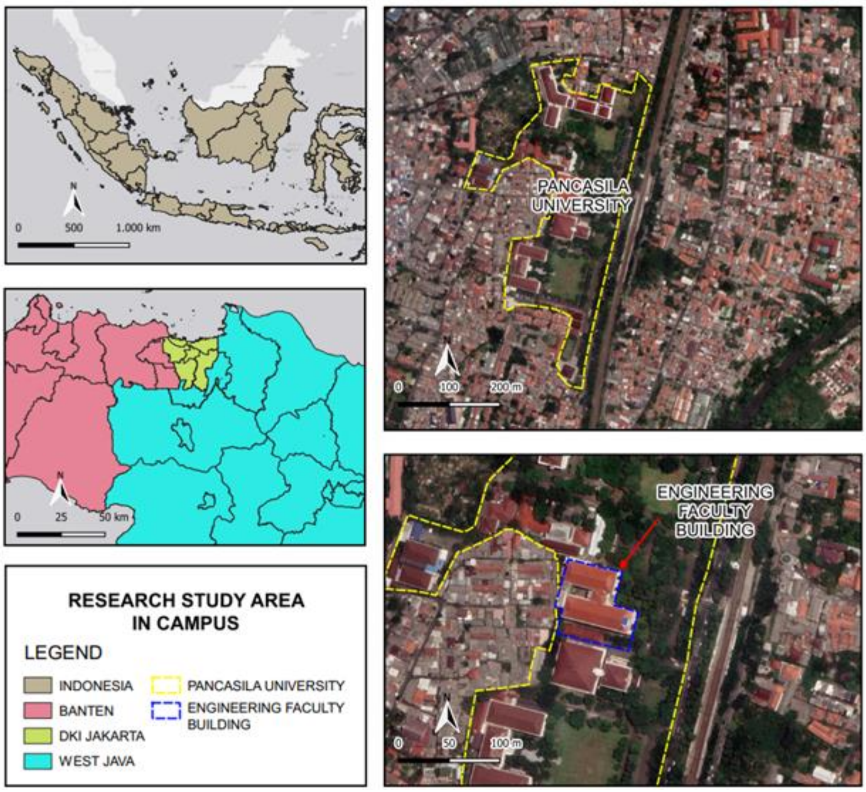


Figure 1. Research study area.

Data Collection

The primary and secondary data were used in this study. Primary data are in the form of measurement data for existing drainage channels at the study location, so the width and height of the existing channel are obtained for secondary data in the form of rainfall data for the last 10 years obtained from the Universitas

Indonesia (UI) rainfall station and topographic data obtained from the *Badan Informasi Geospasial* (BIG). Rainfall data were obtained from *Balai Besar Wilayah Sungai* (BBWS) The closest rainfall observation station was the University of Indonesia Station. Rainfall data were based on the annual maximum rainfall at the University of Indonesia Station 19 last year from 2003 to 2021. The rainfall data used were the maximum daily rainfall data [16,17]. The method used in this study was observation based on field documentation. The data obtained were rainfall, topography, and channel cross-sections, and they are shown in Table 1.

Table 1. Data collection.

No.	Data	Data source	Information
1	Rainfall data for 2003–2021	The daily rainfall data is obtained from BBWS Ciliwung Cisadane.	Rainfall data calculates the planned flood discharge based on the planned rainfall and rain intensity at the study site.
2	Topography	Direct observation in the field based on Google Earth Pro.	The data obtained are the topography of the research location by checking the elevation of the channel under study to obtain the direction of flow.
3	Channel Cross-section	Direct observation in the field	Direct observation in the field to get the existing value. Existing channels are channels that already exist in the field. Then we look back at the dimensions of the drainage channel, and evaluate the drainage channel.

Data Analysis

The analysis method used in this study was a qualitative descriptive method to analyze and plan a drainage system for flood reduction at the study site. The first step was to conduct a data consistency test on the rainfall data at the study location. The RAPS method (Rescaled Adjusted Partial Sums) is used to test data consistency using data from the station itself. This involves testing the cumulative deviation from the mean divided by the square root of the cumulative deviation from the mean squared.

$$\text{Sk*} = \sum_{i=1}^{k}(Yi - \overline{Y}) \quad (1)$$

where:

k = 1, 2, 3, ……., n

$$\text{Sk**} = \frac{\text{Sk}^*}{Dy} \quad (2)$$

$$\text{Dy2} = \frac{\sum_{1=1}^{k}(Yi-\overline{Y})}{n} \quad (3)$$

The statistical values of Q and R are calculated as follows:

Q: maximum |Sk**| for 0 ≤ k ≤ n

R: maximum Sk** – minimum Sk**

Where: Sk* (absolute deviation), Sk** (data consistency value), n (number of data points), Dy (mean deviation), Q (statistical value)

Q for 0 ≤ k ≤ n

R: statistical value (range)

From the results of the calculation Q/n0.5 dan R/n0.5, the count is smaller than that in Table 2, and then it is declared consistent (acceptable or valid).

Table 2. Value of Q/n0.5 and R/n0.5.

n	Q/n0.5			R/n0.5		
	90%	95%	99%	90%	95%	99%
10	1.05	1.14	1.29	1.21	1.28	1.38
20	1.1	1.22	1.42	1.34	1.43	1.6
30	1.12	1.24	1.48	1.4	1.5	1.7
40	1.14	1.27	1.52	1.44	1.55	1.78
100	1.17	1.29	1.55	1.5	1.62	1.85
	1.22	1.36	1.63	1.62	1.75	2

Frequency analysis where the empirical method is determined by using certain return period data, the maximum rainfall, and the methods used there are four methods, namely the Normal Distribution method, Log-Normal Distribution, Log Person Type III Distribution, Gumbel Distribution. The formula used was as follows:

Normal Distribution

$$XT = X + a.\ Sd \tag{4}$$

Log-Normal Distribusion

$$\text{Log } X_T = \text{Log } \overline{X} + a.\ Sd_{LogX} \tag{5}$$

Log Person III Distribution

$$\text{Log } X_T = \text{Log } \overline{X} + KT.\ Sd_{LogX} \tag{6}$$

Gumbel Distribution

$$XT = \overline{X} + K.\ Sd \tag{7}$$

Where: XT (design rainfall), X (maximum monthly rainfall), Sd (standard deviation), a (Normal and Log-Normal Distribution Coefficients), KT (Frequency Factor, a function of probability or return period), and K (standardized variable for X, depending on the magnitude of G).

The time of concentration (Tc) is the time required to drain water from the most distant point in the flow area to a specified control point downstream of the channel. The time of concentration was calculated using the Kirpich formula as follows:

$$tc = 0.0195L^{0.77}.S^{-0.385} \tag{8}$$

Where: tc (time of concentration), L (channel length) and S (slope).

Planning the construction of a drainage canal requires data on the maximum discharge (Q) of rainwater run off that burdens the drainage canal [18]. Considering that short-term rainfall data are not available, only daily rainfall data are available, and the rainfall intensity can be calculated using the Mononbe Method.

$$I = \frac{R_{24}}{24}\left(\frac{24}{tc}\right)^{\frac{2}{3}} \tag{9}$$

Where: I (Rainfall Intensity) and R_{24} (Design rainfall with return period).

The flow coefficient is defined as the ratio between the peak surface run off and rainfall intensity [19]. The C value is the coefficient according to hassing. Manning's n-coefficient values for various types of complete channels were obtained by sharing the references[20–22]. Analysis of hydrological data is required as a basis for calculations to determine the planned rainfall that occurs in an area based on the desired return period. Hydraulic analysis is required to determine the condition of the cross-section's ability to accommodate the design discharge, among others.

a. Analysis of the canal cross-section of the channel found in the research location and using the formula according to the shape of the cross-section [23].
b. Channel Dimensions: The calculation of channel dimensions uses the continuity formula and Manning formula as follows:

$$Q = A \times V \tag{10}$$

Where: Q (discharg), A (cross-sectional area), and V (velocity), respectively.

The water flow velocity is an important parameter in designing the dimensions of the canal, where the minimum permissible velocity does not cause sedimentation and prevents plant growth in the canal. However, the maximum speed allowed does not cause the grinding of the channel material. The Manning formula was used to calculate the capacity of the drainage channel [24,25]:

$$V = \frac{1}{n}.R^{\frac{2}{3}}.S^{\frac{1}{2}} \tag{11}$$

where n (the manning coefficient), and R (the hydraulic radius, which is the ratio of the cross-sectional area (A) of the channel to the wetted perimeter (P).

c. Design Discharge The method used to calculate rainwater discharge in the channel is a Rational Method. The rational method is widely used to estimate the peak discharge caused by heavy rain in small catchment areas (watersheds) [26,27]. This rational method is based on the following equation:

$$Q = 0.278 \times C \times I \times A \quad (12)$$

where C is (coefficient run off).

d. The guard height or freeboard is the vertical distance from the top of the channel to the water surface under the designed discharge conditions.

These topographical data were used to determine the location of the planning study. These data can be used to identify land-use areas and canal network schemes. The Google Earth Pro software was used.

SNI 03-2453-2002 outlines the specific requirements for infiltration wells [28], encompassing both general principles and technical specifications. In adherence to these standards, infiltration wells should be strategically located on relatively flat terrain, and the water entering the catchment must be unpolluted rainwater [29,30]. Decisions regarding infiltration well placement should prioritize safety in building locations and adhere to local regulations. Any deviations from these requirements require approval from authorized agencies. From a technical standpoint, the groundwater level should be at a minimum of 1.5 meters below the surface during the rainy season, and the soil structure used must demonstrate a permeability of ≥ 2.0 cm/hour [31,32].

The design and type of infiltration wells can take the form of either quadrilateral or cylindrical structures, with a specific depth to ensure that the bottom of the well remains above the groundwater level. The Regional Disaster Management Agency of Jakarta identifies various construction types for infiltration wells, including those without retaining walls and unfilled bottoms; those without retaining walls with bottoms filled with stones and fibers; and those with brick, stone, or stone compartments on the well walls, with the bottom filled with stones and fibers or left empty. In addition, wells utilizing blowing, a rock specifically shaped for good walls, are recognized as a construction option. Adhering to these guidelines ensures effective implementation of infiltration well systems in accordance with safety and environmental considerations.

Results and Discussion

Drainage Network

The Drainage Network Scheme at the Faculty of Engineering can be determined based on the results of direct field observations. The determination of the flow direction is carried out based on the scheme that has been made from the existing channel conditions that have been observed, as shown in Figure 2.

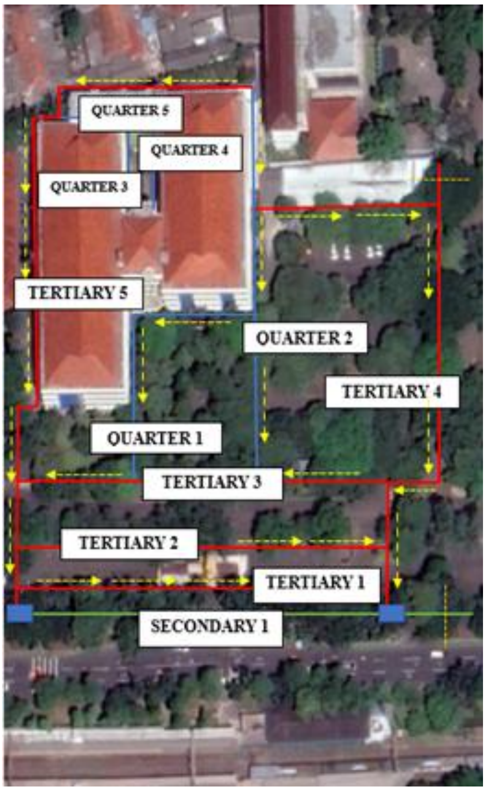


Figure 2. Network schematic and flow direction.

Rainfall Data

Rainfall data were obtained from the BBWS. The closest rainfall observation station was the University of Indonesia Station. Rainfall data were based on the annual maximum rainfall at the University of Indonesia Station for the last 19 years, as shown in Figure 3. For the analysis using the maximum rainfall data, the data in Figure 3 indicate that the highest recorded rainfall was in 2007, reaching 159.5 mm. Coincidentally, in the same year, a major flood occurred in the Jakarta and its vicinity, leading to significant damages [33–35].

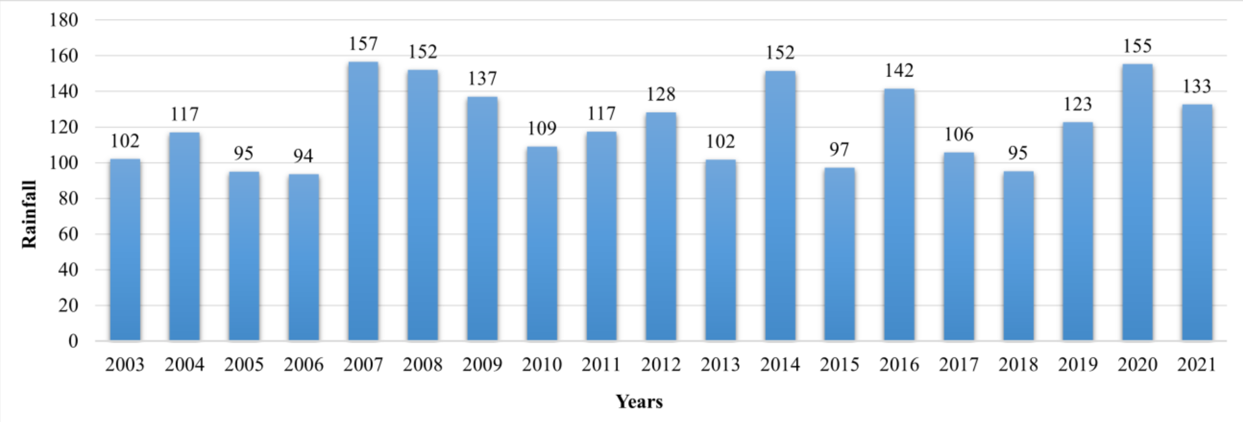


Figure 3. Maximum amount of precipitation.

Data Consistency Test Analysis

Data consistency was tested to validate the accuracy of the data. Rainfall data were considered consistent when the measured and calculated data were aligned correctly with the observed rainfall data. It is essential to conduct consistency testing on rainfall data before proceeding to the next stage of analysis. Using the formula (1) above, the calculated value of Q/n^0.5 is 0.367, and the value of R/n^0.5 is 0.664. These values were then compared to the values in the table. For n = 19, the table values are Q/n^0.5 table = 1.212 and R/n^0.5 table = 1.415. Based on the calculated and table values, it is evident that the calculated values were smaller than the table values. Therefore, the results of the data consistency test were accepted (Table 3).

Table 3. Data consistency test results.

$Q/n^{0.5}$	0.367	Probability 95%	table 1.212 > 0.367 = accepted
$R/n^{0.5}$	0.664	Probability 95%	table 1.415 > 0.664 = accepted

Design Rainfall Analysis

There are 4 methods used for design rainfall analysis: The Normal Distribution Method, Log-Normal Distribution, Log-Person Type III Distribution, and Gumbel Distribution. The design rainfall was chosen based on the goodness-of-fit test results shown in Table 5. It was found that both the Normal and Log-Normal methods met the criteria specified in Table 5. Therefore, the maximum value was selected from these two methods for the 5-year return period (Table 4).

Table 4. Results of design rainfall analysis.

Method	Design rainfall					
	2	5	10	25	50	100
Gumbel	120.882	127.667	132.159	137.836	142.047	146.226
Normal	121.621	126.348	128.824	131.244	133.157	134.733
Log Normal	119.706	139.571	151.260	163.629	174.121	183.264
Log Person Type III	118.270	138.960	152.287	168.793	180.967	193.064

Distribution Test Selection Analysis

Parameter selection from The calculation result of the distribution test to determine the value of the design rainfall that meets the distribution requirements. Based on the values of the skewness coefficient (Cs) and kurtosis coefficient (Ck) [13,36]. The results of the distribution test selection calculations are listed in Table 5.

Table 5. Calculation results from distribution test parameter.

No	Distribution type	Requirement	The calculation results	Conclusion
1	Log Pearson tipe III	Cs ≠ 0	Cs = 0.125	Accepted
2	Gumbel	Cs $\leq$ 1.1396	Cs = 0.125	Accepted
		Ck $\leq$ 5.4002	Ck = −1.475	
3	Normal	Cs ≈ 0	Cs = 0.125	Rejected
		Ck = 3	Ck = −1.475	
4	Log Normal	Cs ≈ 3 Cv + Cv^2 ≈ 3	Cs = 0.125	Rejected
			Ck = −1.475	

From the calculation results, based on the skewness and kurtosis coefficients, it is known that the Pearson type III and Gumbel Log distributions are acceptable and meet the requirements. Design rainfall value is taken from the maximum value between the Pearson type III and Gumbel log methods, so the design rainfall value with 5-year return period is 138.960 mm.

Time of Concentration (Tc)

The concentration time is the amount of time flowing on the surface required for water to reach the maximum discharge from the farthest point of the channel to the point under review. The concentration-time was calculated using the Kirpich formula, and the result was 5 min.

Rainfall Intensity Analysis (i)

The data used to calculate rainfall intensity are measurements of short-term rainfall, typically expressed in units of rainfall intensity (millimeters per hour). These measurements serve as the basis for determining the intensity of precipitation within a specific timeframe. and the results of calculating rainfall intensity are shown in Figure 4.

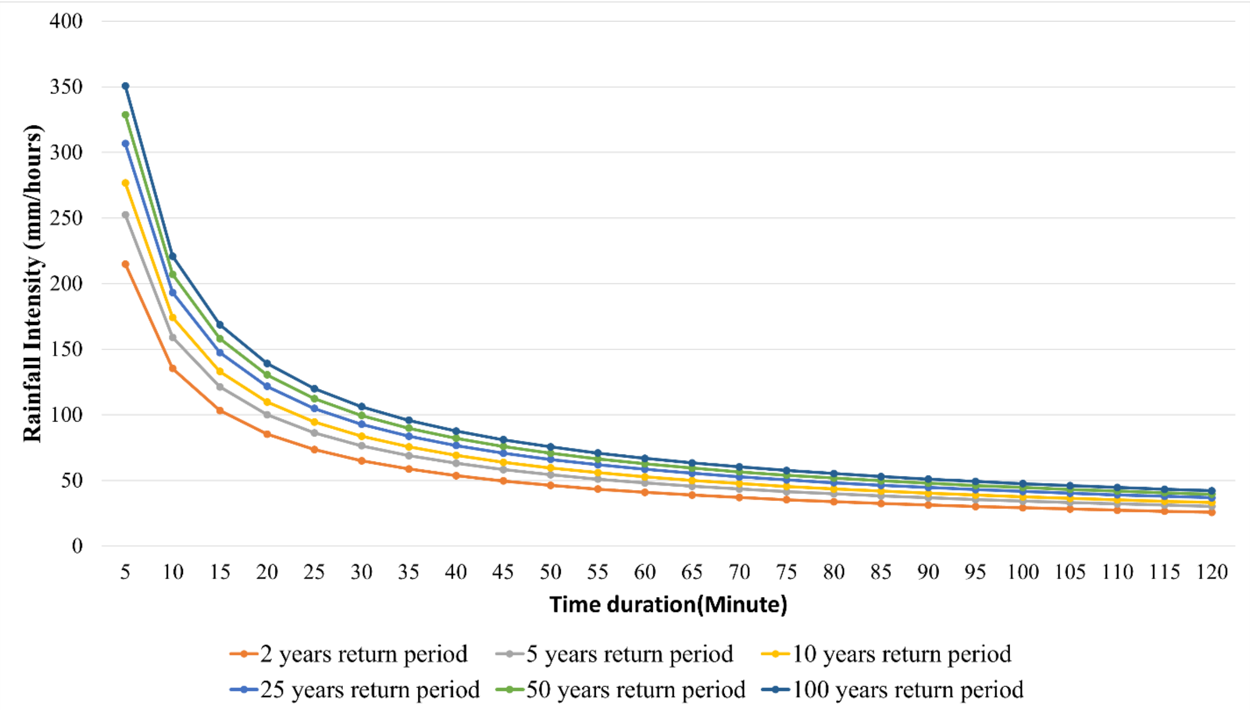


Figure 4. Rainfall intensity diagram.

Run off Coefficient Analysis

The catchment area is highly dependent on surrounding land conditions. The following describes the coefficient of land cover in the case study: The coefficient value of land cover in the form of paving was 0.6, asphalt was 0.8, parks were 0.2 and roofs were 0.8.

Design Flood Analysis

Based on a time of completion (tc) of 5 min, a rainfall intensity value of 253.62 mm/hour was obtained, which can be calculated from the design flood discharge using the rational method; the calculation results for each channel can be seen in Table 6.

Table 6. Design flood discharge.

Channel	Land cover	Run off coefficient (C)	Area (A) m^2	Rainfall intensity (m/Jam)	Flood discharge (Q5 years) (m^3/minute)	Flood discharge (Q5 years) (m^3/s)
Secondary 1	Paving	0.6	12,000	0.252	505.419	1.684
Tertiary 1	Asphalt	0.8	1,100	0.252	61.773	0.205
Tertiary 2	Asphalt	0.8	600	0.252	33.694	0.112
Tertiary 3	Asphalt	0.8	600	0.252	33.694	0.112
Tertiary 4	Asphalt	0.8	3,800	0.252	213.399	0.711
Tertiary 5	Asphalt	0.8	800	0.252	44.926	0.149
Quarter 1	Park	0.2	500	0.252	7.019	0.023
Quarter 2	Park	0.2	1,400	0.252	19.655	0.065
Quarter 3	Roof	0.8	700	0.252	39.310	0.131
Quarter 4	Roof	0.8	800	0.252	44.926	0,.149
Quarter 5	Roof	0.8	200	0.252	11.231	0.0375

Alternatif 1 Redesign Drainage Channels

Based on the results of the calculation of the design flood discharge with a return period of five years, several channels' capacity exceeds the existing capacity based on the design flood discharge (see Table 6). Therefore, the channel was redesigned based on the design of flood discharge. The following conditions were used: a). If the existing flood discharge is greater than the design flood discharge for a return period of 5 years (Qexist > Q5years), then the flood discharge used for channel redesign is the existing flood discharge; b) If the existing flood is smaller than the design flood discharge for a return period of 5 years (Qexist < Q5years), then the flood discharge used for channel redesign is the design flood discharge (Q5 years).

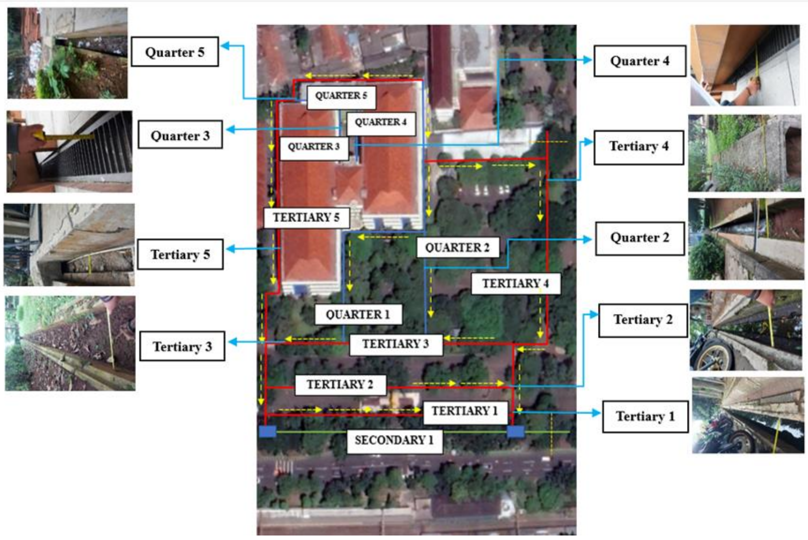


Figure 5. Existing condition of overflowing drainage channels.

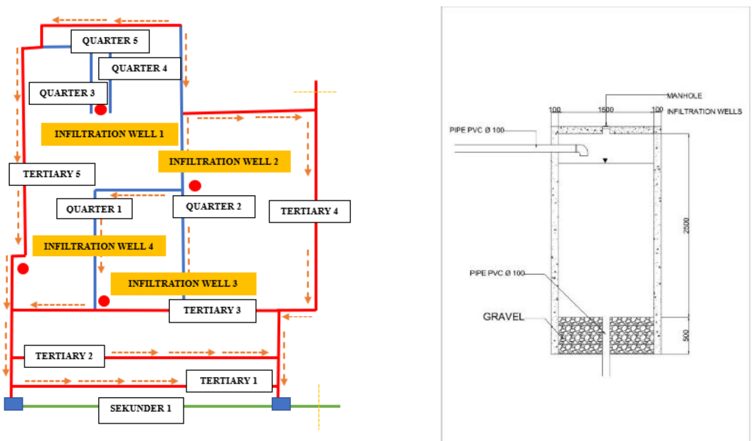
From the above conditions, it is necessary to redesign the channel on the overflow channel, that is, on tertiary 1, tertiary 2, tertiary 3, tertiary 4, tertiary 5, quarter 2, quarter 3, and quarter 4 channels. From Figure 5 Existing flood discharge can be calculated, based on the width of each channel (Table 6). After calculating the redesign of the drainage channel based on the design flood discharge, the channel dimensions were obtained, which can accommodate the redesigned flood discharge based on the design and existing flood discharge (Table 7).

Table 7. Comparison of design flood discharge and existing flood discharges.

Existing dimension			Q existing (m^3/s)	Design flood discharge/Q5 years (m^3/s)	Dimension change		
Channel	High (h)	Wide (b)			High (h)	Wide (b)	Redesign flood discharge (Q) (m^3/s)
Secondary 1	1.5	1.0	1.744	1.685	1.5	1.0	1.744
Tertiary 1	0.4	0.3	0.079	0.206	0.8	0.4	0.206
Tertiary 2	0.4	0.3	0.040	0.112	0.9	0.3	0.112
Tertiary 3	0.3	0.4	0.043	0.112	0.6	0.4	0.112
Tertiary 4	0.7	0.6	0.332	0.711	1.4	0.6	0.711
Tertiary 5	0.5	0.3	0.070	0.150	0.9	0.3	0.150
Quarter 1	0.3	0.3	0.035	0.023	0.3	0.3	0.035
Quarter 2	0.4	0.3	0.047	0.066	0.6	0.3	0.066
Quarter 3	0.1	0.2	0.009	0.131	1.2	0.2	0.131
Quarter 4	0.1	0.2	0.009	0.150	1.4	0.4	0.150
Quarter 5	0.5	0.3	0.070	0.037	0.5	0.3	0.070

Alternative 2 Adding Absorption Wells

From the first alternative, it is known that there are 8 channels that overflow, whose dimensions must be changed; however, the problem has limited land to redesign existing channels. To reduce the run off that occurs by adding infiltration wells, infiltration wells were chosen as a method for dealing with run off from drainage for various reasons, including water conservation, flood reduction, efficiency of existing space, sustainable approach, lower costs, flexibility at various project scales, and suitability to local conditions. The advantages of using infiltration wells include efficiency in utilizing natural resources, lower environmental impacts, and the ability to adapt to the unique conditions of a particular location. at the study location. Infiltration wells are designed based on the amount of flood discharge that overflows in each, from the calculation results, it is known that the total overflow flood discharge which cannot be accommodated by the existing channel is 1,638 m^3/second. The number and dimensions of the infiltration wells can be designed based on the total run off. From the results of field observations and the total overflow flood discharge, it is planned that the number of infiltration wells is 4 infiltration wells, the locations of infiltration wells can be seen in Figure 6 below.



(a) (b)

Figure 6. Drainage system and location of infiltration well (a), detail infiltration well (b).

For infiltration well calculations, based on the predetermined well diameter, namely 1.5 meters with a well height of 2.5 meters, so that the infiltration coefficient and infiltration well volume are obtained to determine the number of infiltration wells needed to handle run off, the results of the infiltration well calculation can be seen in Table 8 below.

Table 8. Result of infiltration wells calculation.

Diameter of the infiltration well (D)	1.5	meters
Infiltration well height (H)	2.5	meters
Coefficient of vertical permability of clay soil (kv) = 2 cm/day	0.48	m^3/m^2/day
Coefficient of vertical permability of clay (kv) = 2*Kv	0.96	m^3/m^2/day
Infiltration well volume (Vs)	4.42	m^3
Base area of the well (Δh)	4.91	m^2
The area of the well wall (Δv)	11.78	m^2
Total area = $\Delta h + \Delta v$	16.68	m^2
Permeability Coefficient (K) = $K = \frac{Kv.\Delta h + Kh.\Delta v}{\Sigma A}$	0.82	
Rain duration	10	minute
	0.17	hour
Well Infiltration Volume (VRsp)	0.09	m^3
One well capacity = Vs + Vrsp	4.51	m^3
Number of wells required = Total volume/Capacity of one well	4	Pieces of infiltration well

From the results of the above calculation with an existing discharge of 1,638 m^3/second, with a storage volume of 16.38 m^3 volume/hour, the dimensions of the infiltration well with a diameter of 1.5 meters and a height of 2.5 meters are obtained, so that the infiltration well is needed to reduce drainage run off discharge of 4 infiltration wells, the location of the infiltration wells can be seen in Figure 6(a) and detailing of infiltration well can be seen in Figure 6(b).

Conclusions

The analysis determined that the planned flood discharge for a return period of 5 years is 138.96 mm, with a rain intensity at a concentration period of 5 minutes calculated to be 0.253 meters per hour. The existing drainage system includes eight channels: Tertiary 1, Tertiary 2, Tertiary 3, Tertiary 4, Tertiary 5, Quarter 2, Quarter 3, and Quarter 4. To manage run off, two flood reduction alternatives were considered. The first involves redesigning the channels based on either the maximum discharge from current flood discharge calculations or the planned flood discharge. The redesigned channel dimensions are listed in Table 7. Significant dimensional changes occurred in Quarter 3 and Quarter 4, where the channel height changed from 10 cm to 1.2 meters in Quarter 3 and 1.3 meters in Quarter 4. In addition to redesigning channels to reduce flood discharge, an analysis was conducted on a second alternative, which involved adding infiltration wells. The design of the infiltration well is based on the run off discharge from the eight channels that overflow, which is 1,638 cubic meters per second. This means that the volume of run off per hour is 1,638 cubic meters. Based on the analysis, the dimensions of the infiltration well were determined, with a planned diameter of 1.5 meters and a depth of 2.5 meters. To accommodate the volume of surface run off, four infiltration wells are needed.

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References

1. Rahman, S.; Khan, M.T.R.; Akib, S.; Bin, N.; Din, C.; Biswas, S.K.; Shirazi, S.M. Sustainability of Rainwater Harvesting System in Terms of Water Quality. *Hindawi* **2014**, *10,* 56–69, doi:http://dx.doi.org/10.1155/2014/721357.
2. Abdillah, A.; Selaman, O.S. Catchment Size, Soil Type and Land Use to Determine the Amount and Likelihood of Flood in the Sarawak Corridor of Renewable Energy (SCORE) Region. *J. Civ. Eng.* **2009**, *1*, 1–6.
3. Molle, F. *River Basin Management and Development*; International Water Management Institute: Kolombo, 2017; ISBN 9781118786352.
4. Esteban, M.D.; Lopez-gutierrez, J.S.; Negro, V. Urban Coastal Flooding and Climate Change Urban Coastal Flooding and Climate Change. *J. Coast. Res.* **2011**, *64*, 205–209.
5. Swain, D.L.; Wing, O.E.J.; Bates, P.D.; Done2, J.M.; Johnson, K.A.; Cameron, D.R. Increased Flood Exposure Due to Climate Change and Population Growth in the United States. *Earth's Future* **2020**, *8*, 1–17, doi:10.1029/2020EF001778.
6. Ariyani, D.; Purwanto, M.Y.J; Euis, S.; Perdinan. Contributing Factor Influencing Flood Disaster Using MICMAC (Ciliwung Watershed Case Study). *Journal of Nat. Resour. Environ. Manag.* **2022**, *12*, 268–280, doi:10.29244/jpsl.12.2.268-280.
7. Buton, S.; Muharyanto, E.A. Evaluation of Drainage Channel Performance on Clove Flower Street. *Uniqbu J. Exact Sci.* **2021**, *2*, 1–7.
8. Wijesekera, S.N.T. Preparation of the Stormwater Drainage Management Plan for Matara Municipal Council. *ENGINEER* **2017**, *44*, 11–29, doi:10.4038/engineer.v44i3.6961.
9. Fraiture, C. D.; Susanto, R.H.; Suryadi, F.X.; Wahyu, H.M.H. Urban Drainage Management and Flood Control Improvement Using the Duflow Case Study: Aur Sub Catchment, Palembang, South Sumatra, Indonesia. *Makara J. Technol.* **2017**, *21*, 1–11, doi:10.7454/mst.v21i2.3085.
10. Tryhorn, L.; Degaetano, A. A Comparison of Techniques for Downscaling Extreme Precipitation over the Northeastern United States. *Int. J. Climatol.* **2011**, *1989*, 1975–1989, doi:10.1002/joc.2208.
11. Bian, G.; Du, J.; Song, M.; Zhang, X.; Zhang, X.; Li, R.; Wu, S.; Duan, Z.; Xu, C. Detection and Attribution of Flood Responses to Precipitation Change and Urbanization: A Case Study in Qinhuai River Basin, Southeast China. *Hydrol. Res.* **2020**, *51*, 1–15, doi:10.2166/nh.2020.063.
12. Jatmiko, D.W. Hydraulics Performance of Coastal Flood Control in Madukoro Area, Semarang City, Indonesia. *J. Civ. Eng. Forum* **2018**, *4*, 189–200.
13. David, A.O.; Agunwamba, J.; Nwaogazie, I.L. Development of Models for Rainfall Intensity- Duration-Frequency for Akure, Development of Models for Rainfall Intensity- Duration-Frequency for Akure, South-West, Nigeria. *Int. J. Environ. Clim. Chang.* **2019**, *9*, 457–466, doi:10.9734/ijecc/2019/v9i830131.
14. Mulyono, D. Analisis Karakteristik Curah Hujan di Wilayah Kabupaten Garut Selatan. *J. Konstr.* **2016**, *12*, 1–9, doi:10.33364/konstruksi/v.12-1.274.
15. Francisco, T.H.S.; Menezes, O.V.C.; Guedes, A.L.A.; Maquera, G.; Neto, D.C. V.; Longo, O.C.; Chinelli, C.K.; Soares, C.A.P. The Main Challenges for Improving Urban Drainage Systems from the Perspective of Brazilian Professionals. *infrastructures MDPI* **2023**, *8*, 1–20.
16. Smith, L.; Liang, Q.; James, P.; Lin, W. Assessing the Utility of Social Media as A Data Source for FLoodrisk Management Using a Real-Time Modelling Framework.Pdf. *Flood Risk Manag.* **2017**, *10*, 370–380, doi:10.1111/jfr3.12154.
17. Breinl, K.; Lun, D.; Müller-Thomy, H.; Blöschl, G. Understanding the Relationship between Rainfall and Flood Probabilities through Combined Intensity-Duration-Frequency Analysis. *J. Hydrol.* **2021**, *602*, 126759, doi:https://doi.org/10.1016/j.jhydrol.2021.126759.
18. Purnama, A.; Saputri, D.E. Studi Kelayakan Saluran Drainase Jalan Sultan Kaharuddin Km. 02 Kabupaten Sumbawa. *J. SAINTEK UNSA* **2016**, *1*, 1–19.
19. Kimi, S. Pengaruh Jenis Dan Kemiringan Dasar Saluran Terhadap Nilai Koefisien C Dengan Persamaan Manning Berdasarkan Hasil Uji Laboratorium. *J. Penelit. Dan Kaji. Tek. Sipil* **2015**, *4*, 1–4.

20. Azmon, B. Manning Coefficient of Roughness – a Case Study along Soreq Stream, 1971–1981. *J. Hydrol.* **1992**, *132*, 361–377.
21. Garrote, J.; Gonz, M.; Guardiola-albert, C.; Herrero, A.D. The Manning's Roughness Coefficient Calibration Method to Improve Flood Hazard Analysis in the Absence of River Bathymetric Data: Application to the Urban Historical Zamora City Centre in Spain. *Appl. Sci.* **2021**, *11*, 1–21.
22. Mohammadpour, R.; Zainalfikry, M.K.; Zakaria, N.A.; Ghani, A.A.; Chan, N.W. Manning's Roughness Coefficient for Ecological Subsurface Channel with Modules. *Int. J. River Basin Manag.* **2020**, *18*, 349–361, doi:10.1080/15715124.2019.1672704.
23. Aldharab, H.; Ali, S.A.; Ikbal, J.; Ghareb, S.A. Analysis of Basin Geometry in Ataq Region, Part of Shabwah Yemen: Using Remote Sensing and Geographic Information System Techniques. *Bull. Pure Appl. Sci.* **2019**, *38*, 1–15, doi:10.5958/2320-3234.2019.00001.5.
24. Abbas, S.A.; Al-aboodi, A.H.; Ibrahim, H.T. Identification of Manning's Coefficient Using HEC-RAS Model: Upstream Al-Amarah Barrage. *Hindawi J. Eng.* **2020**, *1*, 1–7.
25. Nugroho, D.; Leksono, B.; Sholikhah, I. Perencanaan Ulang Sistem Saluran Drainase Di Kecamatan Menganti Kabupaten Gresik. *J. Wahana Tek.* **2021**, *10*, 15–22.
26. Nurhamidin, A.E.; Jasin, M.I.; Halim, F. Analisis Sistem Drainase Kota Tondano (Studi Kasus Kompleks Kantor Bupati Minahasa). *J. Sipil Statik* **2015**, *3*, 599–612.
27. Budianto, M.B.; Yasa, I.W.; Hanifah, L. Analisis Karakteristik Curah Hujan Untuk Pendugaan Debit Puncak Dengan Metode Rasional Di Mataram. *Spektrum Sipil* **2017**, *2*, 137–144.
28. Badan Standarisasi Nasional (BSN). *SNI 03-2453-2002 Tentang Tata Cara Perencanaan Teknik Sumur Resapan Air Hujan Untuk Lahan Pekarangan*; BSN: Jakarta, ID, 2002;
29. Lestari, E.; Makarim, C.A.; Pranoto, W.A. Zero Run-off Concept Application in Reducing Water Surface Volume Zero Run-off Concept Application in Reducing Water Surface Volume. *Mater. Sci. Eng.* **2019**, *508*, 1–7, doi:10.1088/1757-899X/508/1/012019.
30. Ophiyandri, T.; Istijono, B.; Gromiko, A.; Hidayat, B.; Junaidi; Nulrajabmil. Study of Retention and Infiltration Wells as a Result of Resettlement of the Batang Kuranji Upper Watershed. *IOP Conf. Ser. Mater. Sci. Eng.* **2021**, *1041*, 012002, doi:10.1088/1757-899x/1041/1/012002.
31. Palupi, R.; Ramdhan, A.M.; Suwarman, R. Contribution of Groundwater to River Discharge in the Mampang Sub-Watershed. *Int. J. Sci. Soc.* **2023**, *5*, 102–110, doi:10.54783/ijsoc.v5i3.718.
32. Mardiah, A.M.; Ainy, C.N.; Bagus, M.; Harlan, D. Study on the Effectiveness of Infiltration Wells to Reduce Excess Surface Run off in ITB. *MATEC Web Conf.* **2018**, *147*, 1–5, doi:10.1051/matecconf/201814703008.
33. Mohsin, A. Jakarta Under Water: The 2007 Flood and The Debate on Jakarta's Future Water Infrastructure. *J. Wil. dan Lingkung.* **2015**, *3*, 39–58, doi:10.14710/jwl.3.1.39-58.
34. Pratiwi, O.C. Banjir Jakarta 2007, Kegagalan Kolektif Pengelolaan Bumi. *J. Ilmu Sos. dan Ilmu Polit.* **2007**, *11*, 153–286.
35. Wijayanti, P.; Zhu, X.; Hellegers, P.; Budiyono, Y.; van Ierland, E.C. Estimation of River Flood Damages in Jakarta, Indonesia. *Nat. Hazards* **2017**, *86*, 1059–1079, doi:10.1007/s11069-016-2730-1.
36. Adijaya, K.; Prianto, W.; Suripin, S. Penataan Kanal Banjir Timur Semarang. *J. Karya Tek.* **2015**, *4*, 313–323.