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STUDI PENGENDALIAN BANJIR SUNGAI CILEUWIBANGKE AKIBAT PEMBANGUNAN KAWASAN PERUMAHAN DI KOTA BOGOR, JAWA BARAT

STUDY ON FLOOD CONTROL OF THE CILEUWIBANGKE RIVER DUE TO RESIDENTIAL AREA DEVELOPMENT IN BOGOR CITY, WEST JAVA

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ABSTRACT

Bogor City, renowned for its high rainfall value, has experienced rapid economic growth, leading to a substantial surge in population. This, in turn, has prompted extensive development, disrupting land use in the urban area. Consequently, effective area planning incorporating an adequate drainage system is imperative to mitigate flooding. The Cileuwibangke River, located south of Bogor City, traverses a residential area earmarked for conversion into commercial and industrial zones. Therefore, evaluating the floodwater level of the Cileuwibangke River is crucial, both in its existing state and post-construction. This study utilized daily data from the Gadog Rainfall Station and hourly data from the GPM satellite spanning from 2001 to 2020 for rainfall analysis. It revealed that several rain events exceeded the 2, 5, and 10-year return periods. Rainfall-runoff analysis showed that changes in land use resulted in a 35-36% increase in peak flood discharge and a 22-28% increase in runoff volume compared to the existing condition. The findings indicated that the normalization of the river section on the residential area side did not significantly lower the floodwater level, attributed to three broad-dimension culverts. Moreover, strengthening the river's bottom and banks is essential due to the observed hydraulic jump indication. Upstream riverbed protection can be achieved with a 35 m stretch of rock material.

Keywords: *Bogor City, Cileuwibangke River, HEC-HMS, HEC-RAS, and Urban Flood*

ABSTRAK

Kota Bogor, dikenal dengan curah hujannya yang tinggi, mengalami pertumbuhan ekonomi yang tinggi sehingga menyebabkan peningkatan jumlah penduduk yang cukup pesat. Hal ini memicu pembangunan besar-besaran yang mengubah fungsi lahan di kawasan perkotaan. Oleh karena itu, perencanaan kawasan dengan sistem drainase yang memadai diperlukan untuk mencegah terjadinya banjir. Sungai Cileuwibangke, yang terletak di selatan Kota Bogor, mengalir melalui kawasan pemukiman yang akan dikembangkan menjadi kawasan komersial dan industri. Sehingga, perlu dilakukan evaluasi tinggi muka air banjir Sungai Cileuwibangke baik pada kondisi eksisting maupun setelah konstruksi. Studi ini menggunakan data harian Pos Hujan Gadog dan data jam-jaman satelit GPM dari tahun 2001 hingga 2020 untuk melakukan analisis curah hujan. Ditemukan bahwa beberapa kejadian hujan memiliki nilai lebih besar dari hujan rencana periode ulang 2, 5, dan 10 tahun. Berdasarkan analisis curah hujan-limpasan yang dilakukan, diketahui bahwa alih fungsi lahan mengakibatkan peningkatan debit banjir puncak dan volume limpasan masing-masing sebesar 35-36% dan 22-28% dibandingkan dengan kondisi eksisting. Hasil penelitian menunjukkan bahwa normalisasi ruas sungai di sisi pemukiman tidak menurunkan debit banjir secara signifikan, dikarenakan adanya tiga gorong-gorong berdimensi lebar pada kawasan tersebut. Selain itu, perlu dilakukan perkuatan dasar sungai dan bantaran sungai dikarenakan adanya indikasi loncatan hidraulik. Perlindungan dasar sungai dapat dilakukan di hulu dengan menggunakan material batuan sepanjang 35 m.

Keywords: *Banjir Perkotaan, HEC-HMS, HEC-RAS, Kota Bogor, dan Sungai Cileuwibangke*

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INTRODUCTION

Urbanization and rapid population growth in developing countries are significant concerns, as they often result in housing deficits (Monkkonen, 2013). Despite the negative consequences associated with this urban growth, its continuation seems unavoidable (Rustiadi et al., 2021; Grace et al., 2022). While housing stands as one of the fundamental human needs, the influx of rural migrants into cities poses challenges to accessing convenient lodging. Furthermore, the escalating global temperatures, a consequence of global warming, heighten the risk of future disasters, including more frequent heavy rainfall and flooding events (Hidayat & Farihah, 2020). Therefore, the suitability and safety of the living environment are crucial aspects typically considered when choosing a place of residence.

Bogor, known as the City of Rain (Hidayat & Farihah, 2020), is an area in Indonesia characterized by high rainfall values. The economic development in Bogor City and its environs has resulted in a population surge, disrupting the local ecosystem and altering land use for residential purposes. Consequently, a crucial requirement for housing areas in Bogor is the establishment of an effective drainage system to mitigate the risk of flooding in the surrounding areas (Wihaji et al., 2018).

The Cileuwibangke River, located south of Bogor City, flows through residential areas and several prospective commercial and industrial sites. However, a critical consideration arises – the accurate calculation of floodwater levels within the river. This is not merely a concern for the immediate residential areas; rather, it extends to upstream areas that contribute significantly to the natural flow. Notably, Ligtenberg (2017) emphasizes the need for caution regarding flood discharge changes stemming from land use shifts and increasing rainfall in the upstream area. As research unfolds, it becomes clear that upcoming developments in the area must seamlessly integrate with the Cileuwibangke River's flow, safeguarding against drainage system disruptions and potential flooding. The study is a pivotal analysis of existing conditions and those emerging from new developments. This research addresses the immediate concerns of urban expansion and contributes to a broader understanding of managing water systems in evolving landscapes (Alshammari et al., 2023). It complements and extends the previously non-existent discourse on effectively navigating the intricate balance between urban development and environmental resilience.

METHODOLOGY

Study Location

The study was conducted in a residential area in South Bogor, Bogor City, encompassing a total area of 400 hectares. A specific cluster in the northern part, covering 28 hectares, is currently under development, as illustrated in Figure 1. The excess runoff from this particular cluster is slated to be directed towards the nearest river on the east side. It is anticipated that this runoff will escalate in the future with the ongoing development of the northern area.

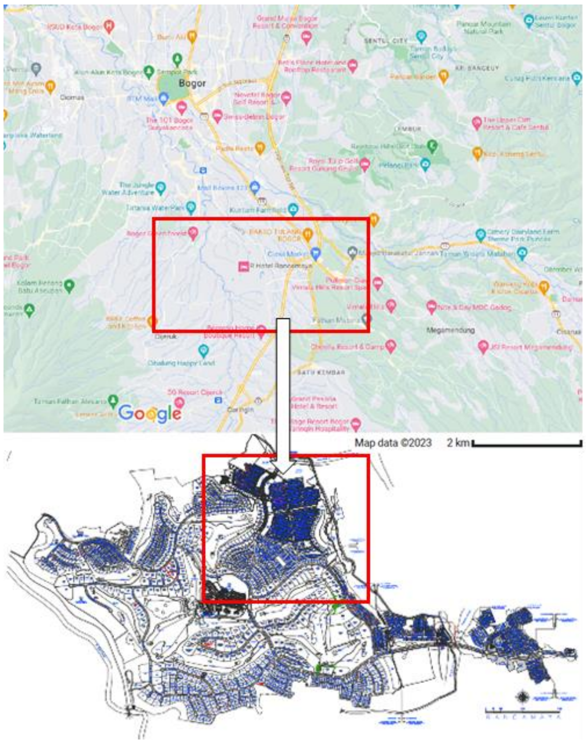


Figure 1 Study Location

Data Availability

In conducting hydrological analysis, this study compiled hourly data from the Global Precipitation Measurement (GPM) satellite and daily rainfall data from BMKG Gadog Rainfall Station spanning from 2001 to 2020, as depicted in Figure 2 (Huffman et al., 2014). GPM provides open-source data with a grid size of 10 x 10 km^2 or 0.1° x 0.1° at the coordinates of 6.65°S, 106.85°E. The locations of the GPM grid and the Gadog Rainfall Station are illustrated in Figure 3. Satellite data offers several advantages in providing rainfall data compared to ground station data, including near real-time availability, swift access, and cost-effectiveness due to its open access policy. However, it is important to note that satellite data has spatial limitations, covering a broader area than the specific watershed under review. To enhance accuracy, satellite data must undergo correction with surface observation data before utilization. (Mamenun et al., 2014).

This study relied on daily rainfall data from the Gadog Rainfall Station and made use of only the hourly rainfall data from the GPM satellite, owing to data limitations. The flow chart diagram of this study is illustrated in Figure 4.

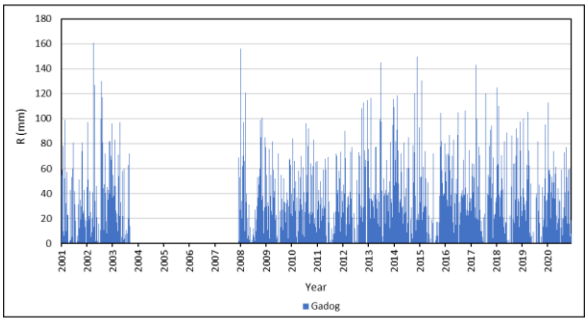


Figure 2 Daily Rainfall data

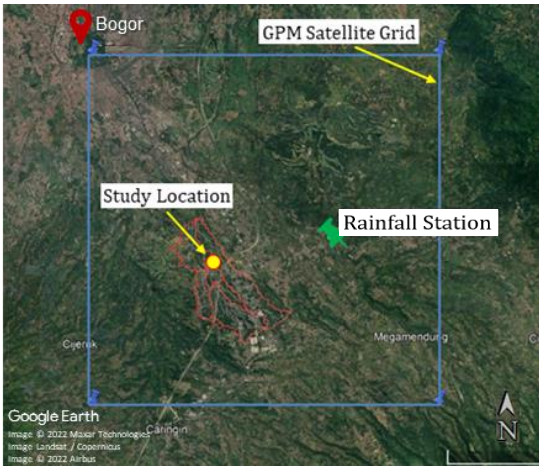


Figure 3 Location of Gadog Rainfall Station and GPM Grid

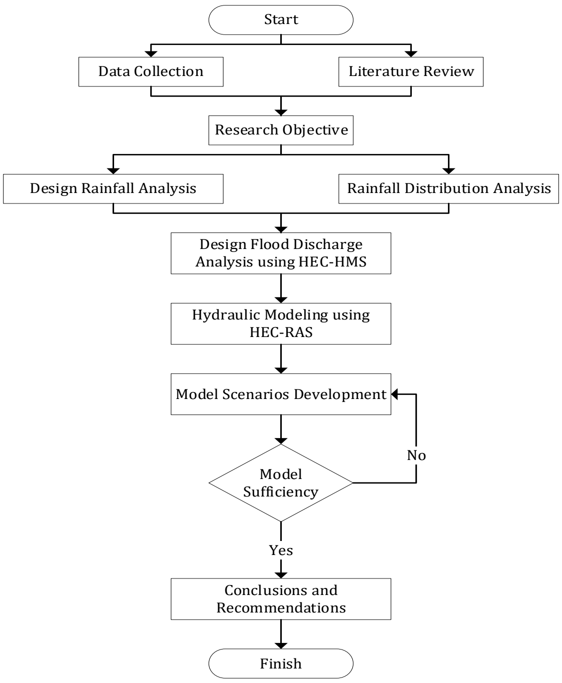


Figure 4 Flow chart diagram

Rainfall Analysis

In this study, the design rainfall was calculated using data from the Gadog Rainfall Station. However, because hourly data was unavailable, the analysis of rainfall distribution was conducted using GPM satellite data. Before incorporating the data into the design rainfall analysis, it was imperative to assess its feasibility. This involved testing for outliers, trends, variance and mean stability, as well as conducting an independence test (Galiatsatou & Prinos, 2007; Zaafrano et al., 2023).

Furthermore, changes in extreme rainfall events are complex in the tropical urban setting, where precipitation activities are influenced by the combined effects of El Nio-Southern Oscillation (ENSO), global warming, and local influences (Li et al., 2018). These extreme values necessitate the development of extensive water structure designs to accommodate the maximum flood discharge. The frequency analysis of rainfall data was employed to establish the relationship between the magnitude of extreme events and their frequency using probability distribution. Notably, there is an inverse relationship between the magnitude of extreme events and the probability of their occurrence (Suripin, 2003; Upomo & Kusumawardani, 2016), expressed by:

$$P(x) = \frac{1}{T} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(1)$$

where:

P(x) : the probability of an event that is equal to or greater than x

T : return period (years)

The probability distribution used in this study was the General Extreme Value (GEV), a probability distribution used to model extreme events such as floods and hurricanes. It is a generalization of the Gumbel, Fréchet, and Weibull distributions and can be used to model both positive and negative extremes (Castillo et al., 2004; Santos et al., 2015). The GEV is defined by its cumulative distribution function (CDF) and is parameterized by three parameters: the location parameter, the scale parameter, and the shape parameter. The PDF equation of the GEV with location parameter μ, scale parameter σ, and form parameter k ≠ 0 is as follows:

$$y = \left(\frac{1}{\sigma}\right)\exp\left(-\left(1 + k\frac{(x-\mu)}{\sigma}\right)^{-\frac{1}{k}}\right)\left(1 + k\frac{(x-\mu)}{\sigma}\right)^{-1-\frac{1}{k}} \ldots\ldots\ldots(2)$$

for $1 + k\frac{(x-\mu)}{\sigma} > 0$, and

$$y = f(x|0,\mu,\sigma) = \left(\frac{1}{\sigma}\right)\exp\left(-\exp\left(-\frac{(x-\mu)}{\sigma}\right) - \frac{(x-\mu)}{\sigma}\right) \ldots\ldots\ldots(3)$$

for k = 0

where the shape parameter (k) determines the skewness of the distribution, the location parameter (σ) determines the location of the distribution on the x-axis, and the scale parameter (μ) determines the spread of the distribution.

Flood Discharge Analysis

Rainfall-runoff analysis in this study utilized the HEC-HMS software, a commonly employed tool for analyzing flood discharge based on rainfall data (US Army Corps of Engineers, 2022; Fahmi et al., 2022). The analysis was conducted using a basin model equipped with hydrological parameters tailored to the characteristics of the watershed. The residential area is passed by the Cileuwibangke River, with the convergence of three tributaries on its upstream side. Downstream of the residential area, the Cileuwibangke River intersects with the Cilulumpang River. The schematization of the hydrological model used for analysis is presented in Figure 5.

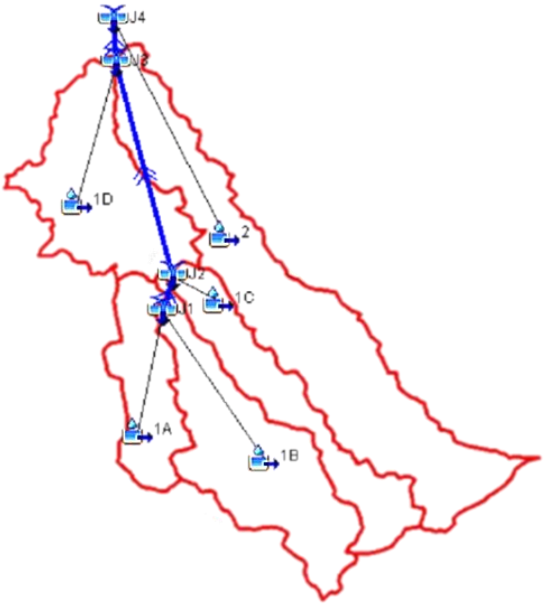


Figure 5 HEC-HMS model schematization

The rainfall-runoff analysis employed the Synthetic Soil Conservation Services Unit Hydrograph (HSS-SCS) method. One crucial parameter in this method is the land cover coefficient, known as the Curve Number (CN) (USDA-SCS, 1972; Ideawati et al., 2015). CN values can be derived based on the land cover type in the study area and are easily adjustable to simulate various scenarios. In this study, the determination of the CN value in existing conditions was based on the land use map, as illustrated in Figure 6 (Badan Informasi Geospasial, 1999). The map was subsequently compared with the existing conditions using Google Earth to discern changes within the past decades. According to the map, residential areas and fields/gardens dominate the watershed. The CN value in the built condition was determined by altering the proportion of land use areas, assuming a change in the area of fields/gardens and paddy fields into residential areas. The CN values utilized in this study are presented in Table 1.

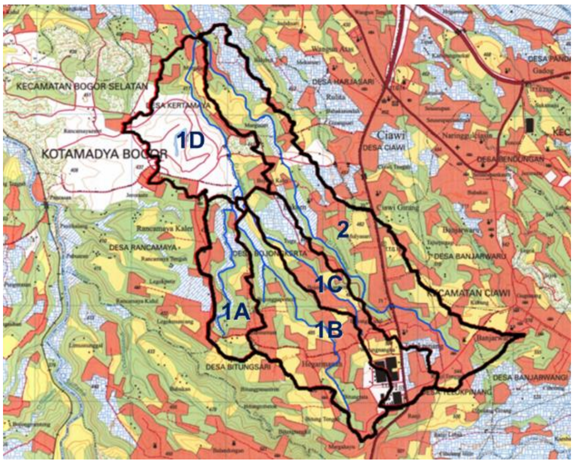


Figure 6 Composition of land use

Table 1 CN values in existing and built conditions

Location	1A	1B	1C	1D	2
Area (ha)	64.4	118.2	75.0	119.6	200.7
CN Existing Condition	66	66	68	73	66
CN Built Condition	73	73	74	74	73

Hydraulic Analysis

River geometry data were acquired through direct field measurements for the Cileuwibangke river section. These measurements were conducted in the upper reaches of the Cileuwibangke River, directly adjacent to the area of interest. The location of the river segment measurements extends from Sta. 0+00 (6.66°S, 106.83°E) to Sta 1+500 (6.65°S, 106.83°E), as depicted in Figure 7. The measurement results cover a crosswise span of 1525 m, illustrated in Figure 8. Several measurement locations, indicated by hills, reveal the presence of three culverts, namely C1, C2, and C3, as shown in Figure 9.

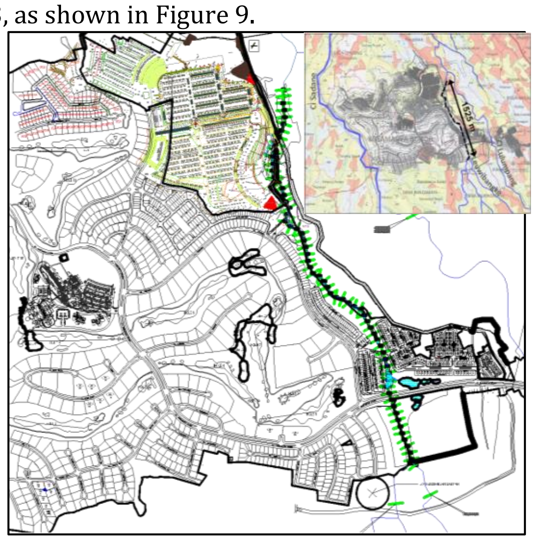


Figure 7 Locations of river geometry measurement

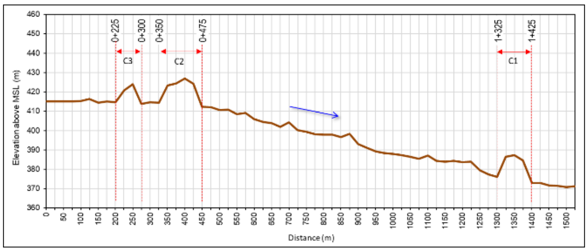


Figure 8 Measurement of the Cileuwibangke River

Figure 9 Location of Culverts

The C1 culvert comprises two square culverts of different sizes. The dimensions of the first culvert on the upstream side are 2.50 m x 2.65 m, while on the downstream side, the dimensions are 1.80 m top width, 1.43 m bottom width, and 2.70 m in height. The dimensions of the second culvert on the upstream side are 2.25 m x 2.65 m, while on the downstream side, the dimensions include a top width of 2.50 m, a bottom width of 2.20 m, and a height of 2.20 m. The base elevation on the upstream side of the two culverts is +376.13 m, whereas the downstream elevation of the first culvert is +374.90 m, and the second culvert is +375.40 m. The water falls freely into the river downstream of the culvert, allowing the neglect of the backwater effect from the downstream area.

Meanwhile, the C2 culvert comprises two circular parallel culverts with a diameter of 2.1 m. The base elevation on the upstream side of the culvert is +413.7 m, and on the downstream side, it is +412.623 m, with a length of 125 meters. In the upstream part of the C2 culvert, sedimentation is present with a height of approximately ±30 cm.

Similarly, the C3 culvert consists of two parallel rectangular culverts with dimensions of 2.5 m x 3.0 m. The base elevation on the upstream side of the culvert is +415.071 m, and on the downstream side, it is +414.844 m, with a length of 75 meters.

On the river section between the C3 and C2 culverts, there is a weir with a height of 1 meter, as illustrated in Figure 10. The weir is positioned downstream of the C3 culvert, just before entering the C2 culvert. The upstream side of the weir has accumulated sediment.

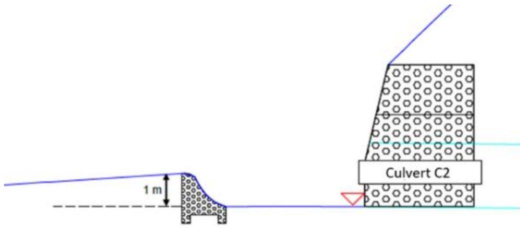


Figure 10 Weir between C2 and C3

Hydraulic analysis of the Cileuwibangke River was conducted using the HEC-RAS hydraulic model. The data obtained and the results of the hydrological modeling served as inputs to the HEC-RAS model for evaluating water flows in the Cileuwibangke River during each period of the design flood discharge. The hydraulic analysis was performed in one dimension, with the model scheme based on the river geometry measurement results, as depicted in Figure 11. In HEC-RAS, several model parameters must be determined as input for a comprehensive hydraulic analysis (US Army Corps of Engineers, 2016). According to the location survey, soil constitutes the riverbed. Using Manning roughness values, the river roughness is determined to be 0.04 (Chow, 1959).

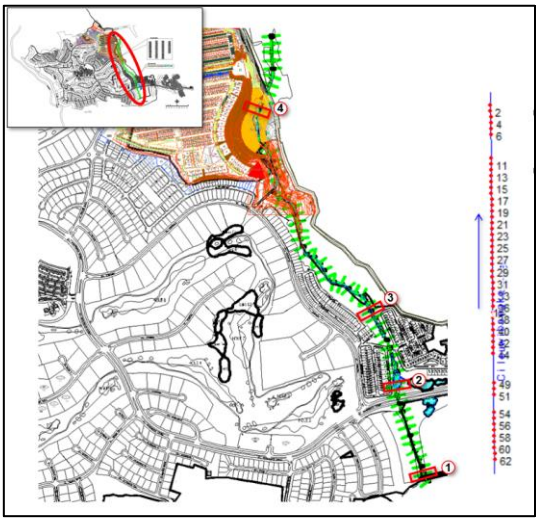


Figure 11 HEC-RAS model schematization

Hydraulic modeling was conducted under both steady and unsteady conditions, utilizing time series discharge data. The simulation's boundary conditions involved flood discharge for the upstream and normal depth for the downstream, with a free fall condition after the culvert. Steady flow analysis was employed to determine the floodwater level, while unsteady flow analysis was used to ascertain the time and duration of flood events. All analyses were carried out with discharge values in both existing and built conditions to discern the impact of area development. Given the absence of discharge and water level records for model calibration and validation, the bank full capacity at the 2-year return period of flood discharge was employed to validate the approximation of the existing condition.

RESULTS AND DISCUSSION

Design Rainfall

The design rainfall analysis involved a frequency analysis, scrutinizing the smallest deviation among various existing probability distributions. The determination of the design rainfall value was then derived from the GEV probability distribution. The assessment of recorded annual maximum daily rainfall data reveals that multiple rain events exceeded the thresholds for the 2, 5, and 10-year return periods, as depicted in Figure 12. Consequently, the evaluation of floodwater levels in the Cileuwibangke River concentrated on discharge resulting from rain events with return periods of 2, 10, and 25 years.

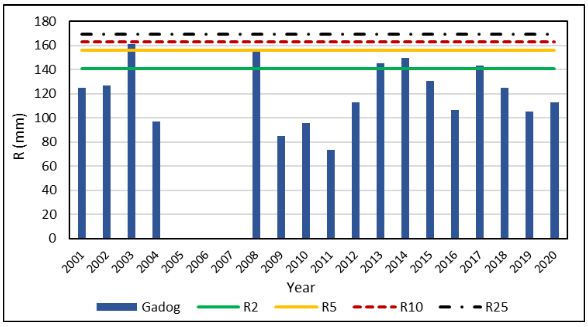


Figure 12 Comparison of maximum annual rainfall and the design rainfall

Design Flood Discharge

The temporal distribution of extreme rainfall was established by identifying extreme rain events recorded in the GPM data. From this record, it was observed that there were 7 extreme rainfall events in the past 20 years, each with a value exceeding 100 mm. Notably, these extreme rain events predominantly occurred over durations of 5, 6, or 7 hours. To analyze the rainfall distribution, the average value for each duration was calculated, as depicted in Figure 13.

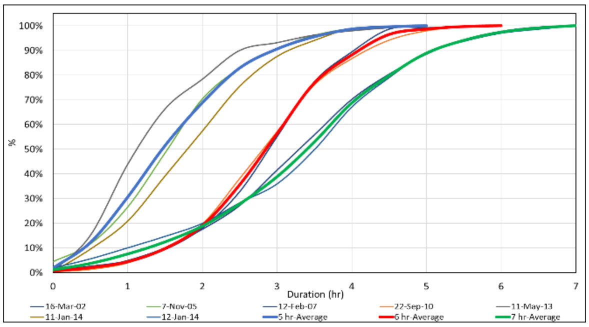


Figure 13 Cumulative rain of extreme rain events

In this study, the temporal distribution of rain was determined based on a 6-hour duration, influenced by the presence of two consecutive peak values, constituting 21% of the rain distribution, as illustrated in Figure 14. Furthermore, modeling results indicated that rain distributed over 6 hours produced the most significant discharge value, despite the peak time occurring later than rain distributed over 5 hours, as demonstrated in Figure 15.

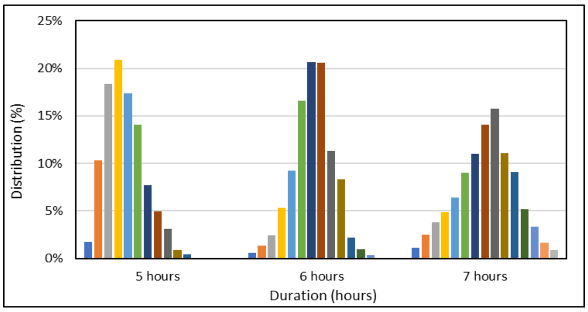


Figure 14 Temporal distribution of rainfall

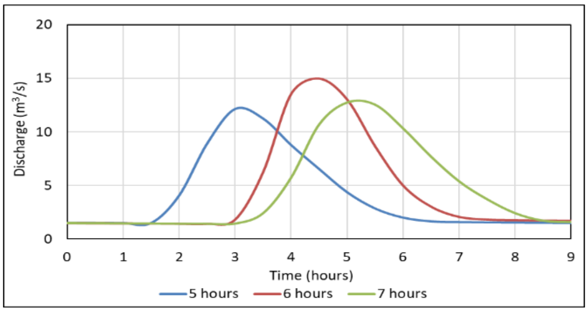


Figure 15 Flood discharge on various rain distribution

Based on the schematization of the hydrological model along with the parameters that have been determined previously, an analysis of the design flood discharge for 2, 10, and 25 years return periods was carried out. The simulation results of the design flood discharge comparison at downstream (J4) (as can be seen in Figure 5) in the existing and the built conditions are shown in Figure 16. It can be seen that the discharge from the upstream area reaches its peak within 4 hours after the rain event occurs. There is an increase of 35-36% in peak discharge and 22-28% in runoff volume at the river downstream as seen in Table 2.

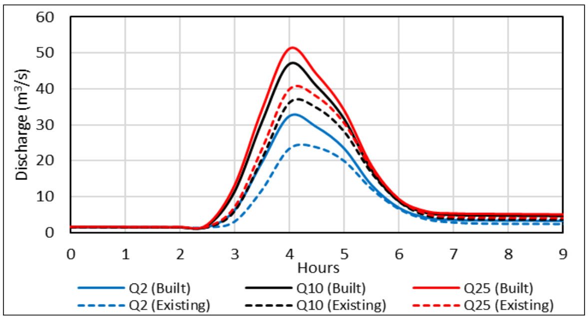


Figure 16 Design discharge at the downstream in existing and built conditions

Table 2 Peak discharge and runoff volume increase

Return Period	Peak Discharge Increase	Runoff Volume increase	Time to Peak (hr)	
			Existing	Built
2	36%	28%		
10	35%	24%		
25	35%	23%	4	4
50	35%	62%		
100	35%	22%		

Hydraulic Analysis

In this study, the hydraulic modeling was done under three scenarios:

- 1st scenario: the river is equipped with culverts and a weir;
- 2nd scenario: the river is equipped with culverts, a weir, and a designed cross-section in the river downstream; and
- 3rd scenario: the 2nd scenario with the addition of embankment in the upstream part of the river.

In the 1st scenario, the analysis was carried out by incorporating the culverts into the existing channel hydraulic model. The existence of the culverts causes backwater to occur, which raises the water level in the upstream part of the culverts. Moreover, the culverts also cause an increase in the water level at the culvert entries. In more detail, the longitudinal section of the river in various return periods for the 1st scenario are shown in Figure 17 (a).

The 2nd scenario evaluates the flood attenuation due to river normalization, which is located upstream of the C1 culvert. The modeling results show that the normalization does not significantly lower the water level as shown in Figure 17 (b). This outcome is due to the large capacity of the culverts downstream of the flood reservoir.

As for the 3rd scenario, there was overflow of water in the section between Sta. 0+025 to Sta. 0+250 when it rained. Therefore, this scenario was done to control runoff using an embankment upstream. The embankment is required to ensure no overflow occurs upstream of the residential areas where the ground is lower than the riverbanks. In this scenario, a flood embankment on the left side of the Sta. 0+025 to Sta. 0+250 (225 m long) is added. The height of the embankment is planned to be around 1.5 m (varies according to the elevation) by maintaining the top of the embankment at an elevation of +418.32 m. The embankment is designed to be constructed at a distance of 11.10 to 14.45 meters from the left bank of the river. The comparison of the upstream condition before (2nd scenario) and after the construction of the embankment (3rd scenario) can be seen in Figure 18. The design location of the levee is on the purple line in Figure 19.

Flood Arrival Time and Duration

In order to analyze when and how long the flood will occur, a hydraulic analysis was carried out on unsteady flow conditions. The analysis was done with return periods of 2, 10, and 25 years in the condition of the built area. This selection is based on the Minister of Public Works Regulation of the Republic of Indonesia No. 12 of 2014 Concerning the Implementation of Urban Drainage Systems. For this study, the appropriate return period is 5-10 years. However, because the developer does not yet have a further land development plan, this study considers a more extensive return period to determine the impact that will ensue on extreme flood discharges and reduce risk (Table 3).

Table 3 Peak discharge, volume, and time to peak

Return Period	Peak Discharge (m^3/s)	Runoff Volume (1000 m^3)	Time to peak (hour)
2	15.14	146.54	4
10	21.74	205.40	4
25	23.23	222.33	4
50	24.59	231.21	4
100	25.32	237.85	4

The hydraulic analysis under unsteady flow conditions resulted in a reduction in discharge, with the peak discharge value being smaller than the modeling results obtained under steady flow conditions. A comparative assessment was conducted between the modeling outcomes in unsteady flow conditions and those in steady flow conditions, focusing on the maximum floodwater level observed at Sta 1+325. At this specific location, the disparity in water levels ranged from 4 to 5 cm. Based on the analysis results, flooding would occur for ± 4 hours. The analysis results on unsteady flow conditions are briefly shown in Table 4.

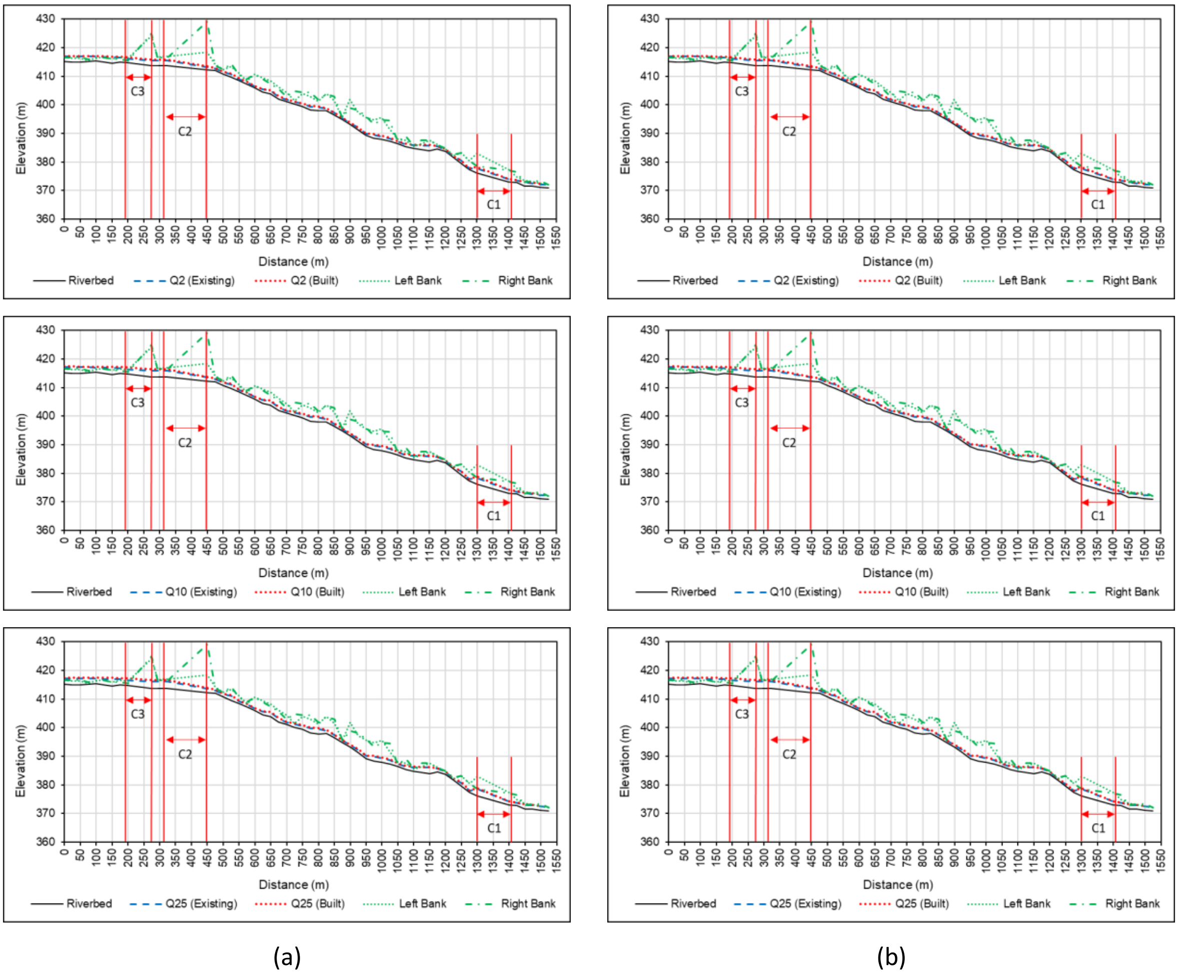


Figure 17 Longitudinal section of the river in various return periods for the 1st scenario (a) and 2nd scenario (b)

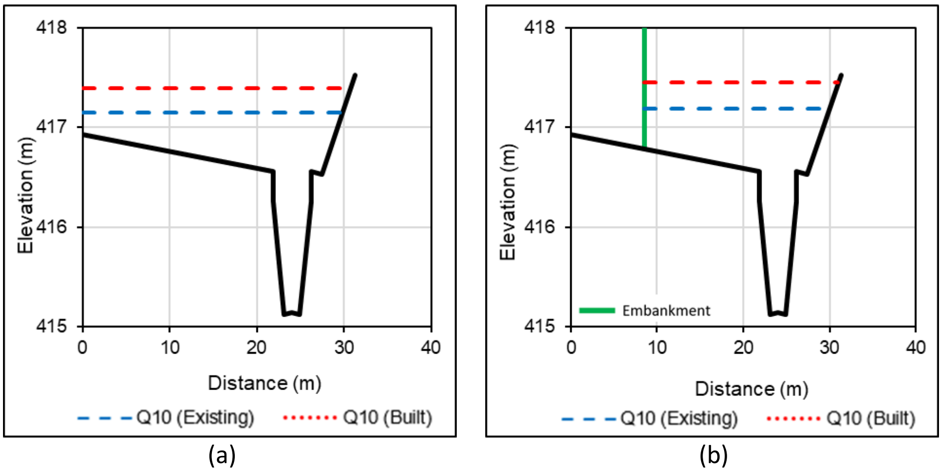


Figure 18 Comparison of the upstream condition before (a) and after the construction of the embankment (b) in Sta 0+025

Figure 19 The location of the flood embankment

Table 4 The comparison of the steady and unsteady flow analysis

Return Period	Max Flood Level (m) Unsteady Flow	Max Flood Level (m) Steady Flow	Difference (cm)
2	378.19	378.22	3
10	378.74	378.79	5
25	378.89	378.94	5
50	378.96	379.02	6
100	379.03	379.07	4

Figure 20 shows the maximum water level simulated by HEC-RAS model. Based on the results obtained, it was observed that the initial floodwater level occurred approximately 2 hours and 40 minutes after the onset of the rain event at Sta. 1+350. The water level gradually rose until it peaked after 4 hours and 10 minutes, subsequently returning to its initial state. The increase in water level due to flooding occurred within a ± 3 hours and 40 minutes timeframe.

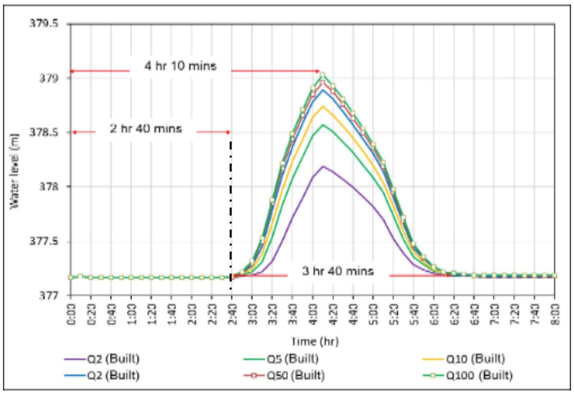


Figure 20 Flood water level at STA 1+325

Flood Control of the Cileuwibangke River

Flood control becomes imperative to manage the occurrences of flooding along the Cileuwibangke River, particularly in areas intersecting with residential zones. The modification of the Cileuwibangke River aims to curtail the height of flooding, preventing inundation of the surrounding areas.

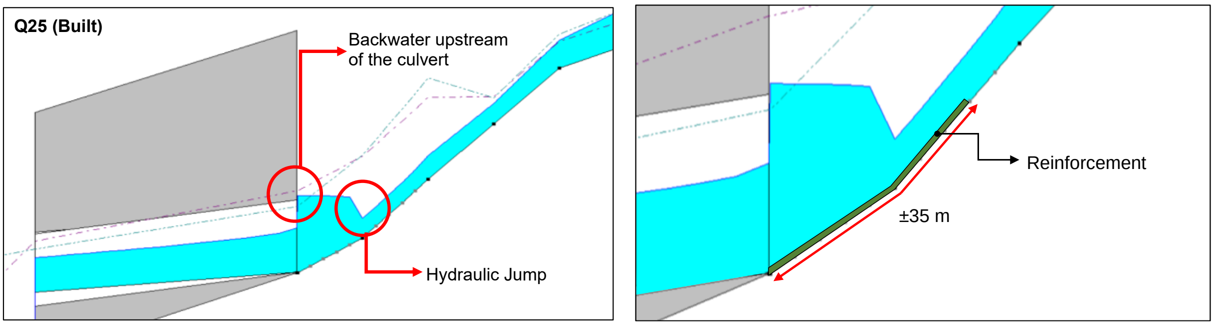
However, according to the analysis conducted using the HEC-RAS model, the normalization of the river channel on the upstream side of the C1 culvert showed no impact on lowering the floodwater level, primarily due to the substantial capacity of the culvert. The results of the analysis at the normalization location are detailed in Table 5.

Table 5 Water level (msl) comparison with and without culvert and normalization

Return Period	Without Culvert Existing	Without Culvert Built	With Culvert Existing	With Culvert Built	With Culvert & Normalization Existing	With Culvert & Normalization Built
2	376.75	376.88	378.79	379.42	378.49	379.13
5	376.87	377.01	379.32	380.11	379.02	380.04
10	376.92	377.06	379.66	380.2	379.36	380.14
25	376.96	377.11	380.00	380.27	379.70	380.21

Upon closer examination of the normalization area, a hydraulic jump was observed before entering the C1 culvert as shown in Figure 21 (a). This occurrence stems from the relatively steep slope of the channel on the upstream side of culvert C1, inducing a supercritical condition that meets the culvert with a comparably gentle slope. The modeling results reveal a maximum flow velocity of 5.23 m/s, emphasizing the necessity to ensure the safety of the riverbed and banks to prevent erosion and landslides. To address this, riverbed protection measures are recommended, involving the use of rock material (unit weight ±50 kg per rock) for a minimum span of 35 meters from the C1 culvert inlet upstream as shown in Figure 21 (b).

In addition to the normalization efforts, flood control measures are deemed necessary upstream of the Cileuwibangke River. The hydraulic analysis results (scenarios 1 and 2) indicate overflow in the left upstream section of the river. Consequently, plans include the construction of an embankment to regulate flooding and prevent inundation of the area.



(a) (b)

Figure 21 Location of the hydraulic jump upstream of the culvert (a) and reinforcement location recommendation (b)

The flood embankment is designed at a distance of 11.10 to 14.45 meters from the left bank of the river, with the top elevation set at +418.32 meters (embankment height ± 1.5 m). A freeboard of 80 cm is incorporated to account for the 25-year return period. With the addition of the embankment, the river's floodwater level would fluctuate with a maximum increase of 6 cm. The disparity in water levels before and after the embankment, particularly at Sta 0+025, is illustrated in Table 6.

Table 6 Water level comparison with and without embankment

Return Period	Without Embankment (m)		With Embankment (m)		With Level Increase (cm)	
	Existing	Built	Existing	Built	Existing	Built
2	416.95	417.12	416.97	417.16	2	4
5	417.09	417.26	417.13	417.32	4	6
10	417.15	417.40	417.19	417.45	4	5
25	417.20	417.48	417.25	417.53	5	5

CONCLUSIONS

Based on the analysis, changes in land use resulted in a 35-36% increase in peak flood discharge and a 22-28% increase in runoff volume downstream of the river. The normalization of the Cileuwibangke River in the residential area has a minor impact on lowering the water level in the river due to the broad dimensions of the culverts. Given the high velocity on the upstream side of the C1 culvert, it is necessary to reinforce the river's bottom and riverbanks due to the indication of a hydraulic jump. Additionally, an embankment can be employed to address flooding on the left side of the river from Sta. 0+025 to Sta. 0+250. However, it should be noted that the construction of the embankment will raise the water level in the river by 2-6 cm.

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