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Evaluating the Effectiveness of Flood Control Scenarios in Reducing Flood Hazard Index Levels in the Ciletuh River, Indonesia

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ABSTRACT

Flood hazards in tropical watersheds are increasing due to extreme rainfall and rapid hydrological responses, particularly in low-lying agricultural regions. The downstream Ciletuh River in Ciemas District frequently experiences inundation that causes significant flood impacts. This study evaluates the effectiveness of structural flood control scenarios in reducing inundation extent, depth, flow velocity, and flood hazard levels. Hydrological analysis was conducted using rainfall frequency analysis and the SCS Unit Hydrograph, while hydraulic simulations were performed using coupled one-dimensional and two-dimensional HEC-RAS models for 2-, 5-, 10-, and 25-year return periods. Under the Q25 condition, the existing inundation area of 584 ha was reduced by 26% under the levee scenario and 32% under the normalization scenario. The combined levee–normalization scenario achieved the highest reduction of 74%, although this value is subject to uncertainty due to methodological limitations. The combined scenario also generated the highest peak velocity of 4.06 m/s, indicating improved hydraulic conveyance efficiency. This study integrates the BNPB Flood Hazard Index framework to support flood mitigation planning in the downstream Ciletuh Watershed.

1. INTRODUCTION

Flooding is a hydrometeorological disaster that has shown increasing frequency and intensity in tropical regions (Tabari, 2020). The IPCC reports that the intensification of the hydrological cycle due to climate change increases the occurrence of extreme rainfall and surface runoff risk (IPCC, 2023). The intensification of extreme rainfall directly increases peak river discharge and the likelihood of flows exceeding channel capacity (Alifu *et al.*, 2022). In small to medium watersheds with steep topography, hydrological responses occur rapidly because high-intensity rainfall is quickly converted into significant peak discharge (Blöschl *et al.*, 2019). Water levels and flood inundation distribution are influenced by channel geometry, bed slope, and channel roughness (Brunner, 2024). Therefore, flood evaluation requires the integration of hydrological and hydraulic analyses within a consistent modeling framework.

The Ciletuh Watershed, covering approximately 192.76 km², is located in Sukabumi Regency, with this study focusing on the downstream reach of the Ciletuh River in Ciemas District. This area is relatively flat and dominated by agricultural land and settlements. Data from the West Java Provincial Water Resources Agency recorded a maximum discharge of 281.64 m³/s in 2024. The West Java Disaster Management Agency (BPBD) reported 104 affected residents, damage to 32 houses and 7 public facilities, and approximately 100 ha of inundated agricultural land (BPBD, 2025).

Historical flood records for the 2020–2025 period show significant fluctuations between maximum rainfall, peak discharge, and flood occurrence frequency, as presented in Figure 1. This pattern indicates the watershed's hydrological sensitivity to extreme rainfall and the limited hydraulic capacity of the downstream segment (Merz *et al.*, 2021).

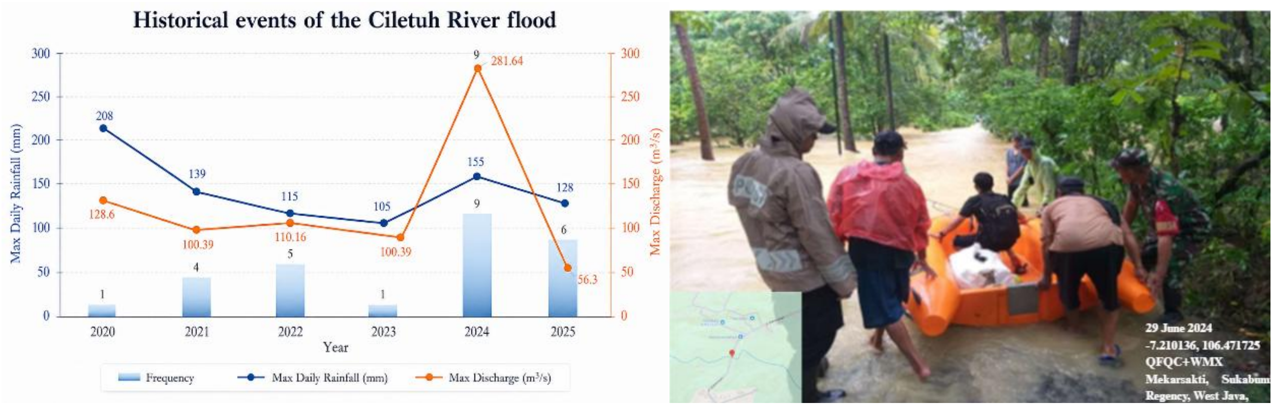


Figure 1. Historical flood events of the Ciletuh River in Ciemas District

Most of the downstream area of Ciemas District consists of productive rice fields; thus, flooding causes crop damage, disrupts planting seasons, and generates potential economic losses for local communities (Allaire, 2018). Although levees and channel normalization have been proposed as flood mitigation measures for the downstream Ciletuh River, their hydraulic effectiveness has not yet been quantitatively evaluated using an integrated hydrological–hydraulic modeling framework. Therefore, these interventions were assessed in this study as simulated structural mitigation scenarios.

Recent advances in hydrological and hydraulic modeling enable integrated flood assessment through coupled one-dimensional (1D) and two-dimensional (2D) simulations. One-dimensional models effectively represent channel flow and water surface profiles, while two-dimensional models improve the spatial representation of flood extent, depth, and flow dynamics (Teng *et al.*, 2017). However, limited studies have integrated multi-distribution rainfall frequency analysis, rainfall–runoff transformation, coupled 1D/2D HEC-RAS modeling, and flood hazard classification based on the Regulation of the Head of the Indonesian National Disaster Management Agency (BNPB) Number 2 of 2012, which provides the criteria for classifying flood hazard index levels, within a unified comparative framework for evaluating structural mitigation scenarios. Therefore, this study develops an integrated modeling framework to assess the effectiveness of levee construction, channel normalization, and combined structural measures in reducing flood inundation extent, flood depth, and flood hazard classes in the downstream Ciletuh River.

2. MATERIALS AND METHODS

2.1. Study Location dan Data

This study employed a quantitative approach by integrating hydrological analysis, including outlier testing, data consistency analysis using the Rescaled Adjusted Partial Sums (RAPS) method, and rainfall frequency distribution analysis supported by statistical parameters such as mean, standard deviation, skewness, and kurtosis to estimate design rainfall for various return periods, with two-dimensional hydraulic modeling to evaluate flood hazard levels and the effectiveness of flood control scenarios in the Ciletuh Watershed, Sukabumi Regency. The analysis focused on the downstream segment of the Ciletuh River (±5.75 km) as the main channel, as well as its tributaries, the Cikalong River (±6.81 km) and the Cidahon River (±5.74 km), as shown in Figure 2.

The watershed boundary, river network, and study segments are presented in Figure 3. The Ciletuh Watershed covers an area of approximately 192.76 km², while the study area located in the downstream part of Ciemas District covers approximately 16 km². The data utilized in this study included:

a. Daily and annual maximum river discharge data for the period 2004–2024 obtained from the West Java Provincial Water Resources Agency (DSDA, 2025);
b. Annual maximum rainfall data in the Ciletuh Watershed for the period 2004–2024 obtained from the Ciletuh Rainfall Station (7.207500° S, 106.536944° E) and Maranginan Rainfall Station (7.238000° S, 106.532000° E), provided by the West Java Provincial Water Resources Agency (DSDA, 2025);
c. Surveyed river cross-sectional data for one-dimensional (1D) calibration in HEC-RAS;
d. Digital Elevation Model (DEM) data for two-dimensional (2D) flood inundation simulations in HEC-RAS;
e. Land use data obtained from the Ministry of Environment and Forestry (KLHK, 2022);
f. Observed flood event data for model calibration and validation.
g. Areal rainfall was calculated using the Thiessen Polygon method, as it is suitable for limited and unevenly distributed rainfall stations.

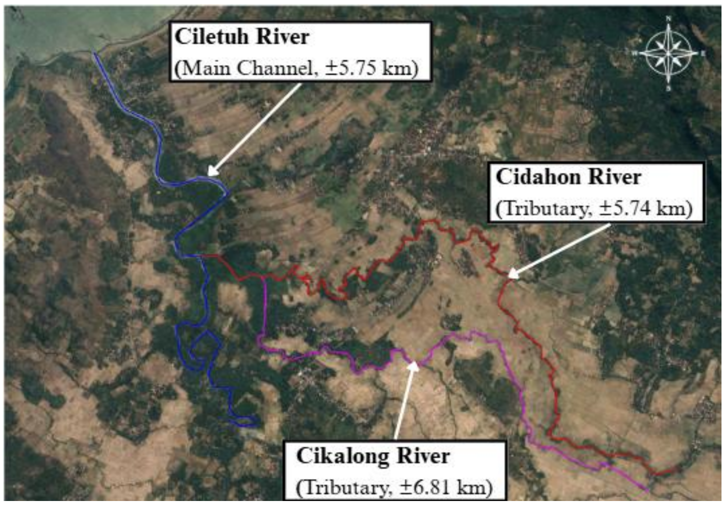
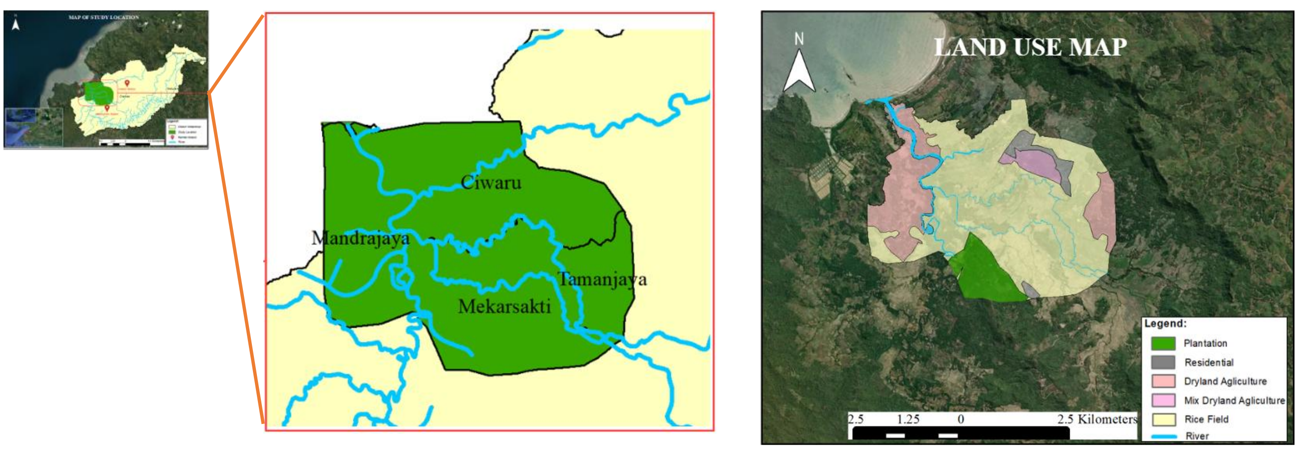


Figure 2. Main channel and tributary river segments in the study area



(a) (b)

Figure 3. Ciletuh watershed map: (a) study area location on Ciemas district, (b) Land use map of the study area

2.2. Watershed Characteristics

Morphologically, the Ciletuh Watershed is dominated by hilly to mountainous terrain with gentle to steep slopes extending from the upstream to the midstream areas, forming a dendritic drainage pattern that accelerates flow concentration toward the main river (Obeidat et al., 2021). Steep slopes in the upstream area increase surface runoff and shorten the time of concentration, while the relatively flat downstream area functions as a flow accumulation zone and is therefore more prone to inundation. Land use is predominantly plantation areas in the upstream region (approximately 60%), which may enhance runoff and erosion on sloping terrain, whereas the downstream area is dominated by paddy or rice fields (approximately 38%) (Rogger *et al.*, 2016; KLHK, 2022). Consequently, Ciemas District functions as a flood-receiving area characterized by flat topography and low elevation, which increases its susceptibility to inundation.

The land use distribution within the study area (approximately 16 km²), presented in Figure 3b and Table 1, indicates the dominance of paddy fields in the downstream segment of the Ciletuh River. The relatively flat characteristics and low elevation of paddy fields significantly influence the spatial distribution and depth of inundation in the two-dimensional simulation results.

Table 1. Land use area in the study area

Land Use Types	Area (ha)	Percentage (%)
Plantation	126	8%
Residential	53	3%
Dryland Agriculture	317	20%
Mixed Dryland Agriculture	47	3%
Paddy / Rice Field	1.053	66%

2.3. Hydrological Analysis

2.3.1. Rainfall Frequency Analysis

Frequency analysis was conducted to determine design rainfall for return periods of 2, 5, 10, and 25 years, in accordance with protection standards for residential areas within medium-sized watersheds. The probability distributions tested included Normal, Log-Normal, Log-Pearson Type III, Gumbel, and Generalized Extreme Value (GEV). The best-fit distribution was selected using the Kolmogorov–Smirnov test at a 5% significance level by comparing the maximum deviation between empirical and theoretical distributions. The results indicate that the GEV distribution provides the best fit and was therefore adopted for estimating design rainfall. This approach is consistent with recent probabilistic flood hazard studies, where the GEV distribution was identified as the most suitable model through statistical goodness-of-fit tests and subsequently integrated with HEC-RAS simulations to improve the reliability of flood inundation mapping (Shaikh *et al.*, 2025). The cumulative distribution function of the GEV is expressed as:

$$F(x) = \exp\{-[1 + \xi(\frac{x-\mu}{\sigma})]^{-1/\xi} \quad (1)$$

where μ is location parameter, σ is scala parameter, and ξ is shape parameter. The GEV distribution is widely applied in hydrological extreme event analysis due to its flexibility in representing tail behavior characteristics.

2.3.2. Effective Rainfall (SCS-CN Method)

The design rainfall was subsequently transformed into a hyetograph using the PSA 007 method with an effective duration of 6 hours, selected to reflect the hydrological response characteristics of small to medium-sized watersheds. Effective rainfall was estimated using the Soil Conservation Service–Curve Number (SCS-CN) method. The composite Curve Number (CN) value was determined through spatial overlay of the 2022 land use data and Hydrologic Soil Group C, applying area-weighted averaging. This procedure yielded an average CN value of 77.9 under Antecedent Moisture Condition II (AMC II). The potential maximum retention was calculated as:

$$Q = \frac{(P-0.2S)^2}{P+0.8S}, P > 0.2S \quad (2)$$

where:

$$S = \frac{25400}{CN} - 254 \quad (3)$$

The SCS-CN method was selected due to its suitability for watersheds with limited hydrological data availability and its widespread application in rainfall–runoff modeling studies (Mishra & Singh, 2003).

2.3.3. Rainfall–Runoff Transformation

a. SCS Unit Hydrograph

Effective rainfall derived from the SCS Curve Number method was subsequently transformed into a discharge hydrograph using the Soil Conservation Service Unit Hydrograph (SCS-UH) method. This approach is widely applied to small and medium-sized watersheds because it effectively represents event-based runoff response using relatively simple parameters derived from watershed physical characteristics (USDA-SCS, 1972). The unit hydrograph peak time was calculated using the following equation:

$$T_p = T_{lag} + \frac{D}{2} \tag{4}$$

where T_p is peak time (hour), T_{lag} is lag time (hour), and D is effective rainfall duration (hour). The lag time was calculated as:

$$T_{lag} = 0.6\, T_c \tag{5}$$

where $T_c$is the watershed time of concentration, The peak discharge of the unit hydrograph (Q_p) was calculated as follows:

$$Q_p = \frac{2.08\, A}{T_p} \tag{6}$$

where Q_p is unit hydrograph peak discharge (m³/sec/mm), A is area (km²), and T_p is peak time (hour).

b. Snyder Unit Hydrograph

The Snyder synthetic unit hydrograph estimates peak discharge based on watershed lag characteristics and empirical regional coefficients. The lag time was calculated as:

$$T_{lag} = C_t (L \: x \: L_c)^{0.3} \tag{7}$$

where T_{lag} is lag time (hour), C_t is regional coefficient, L_c is distance from outlet to centroid (km), and L is main stream length (km). The peak discharge was computed using:

$$Q_p = \frac{C_p A}{T_p} \tag{8}$$

where Q_p is unit hydrograph peak discharge (m³/sec/mm), A is area (km²), T_p is peak time (hour), and C_p is peak coefficient (hour).

c. Nakayasu Synthetic Unit Hydrograph

The Nakayasu method estimates flood hydrographs based on watershed area and concentration characteristics. The peak discharge was calculated using:

$$Q_p = \frac{1}{36} \times \frac{A \times R_e}{0.3\, T_p + T_{0.3}} \tag{9}$$

where Q_p is unit hydrograph peak discharge (m³/sec/mm), A is area (km²), T_p is peak time (hour), R_e is effective rainfall (mm), and $T_{0.3}$ is recession time.

The Snyder method estimates the synthetic unit hydrograph based on watershed lag time and empirical coefficients related to basin storage and peak characteristics, whereas the Nakayasu method utilizes watershed area, river length, concentration time, and recession characteristics to construct the hydrograph shape. Key parameters used in the comparative analysis included watershed area, main river length, channel slope, lag time, and concentration time derived from watershed morphometric characteristics.

The design flood hydrograph was generated through the superposition of effective rainfall and the unit hydrograph to obtain total runoff discharge at each time interval. Several rainfall–runoff transformation methods, namely the SCS

Unit Hydrograph, Snyder, and Nakayasu methods, were evaluated by comparing the 2-year return period peak discharge (Q2) with the bankfull discharge derived from cross-sectional analysis and one-dimensional (1D) model calibration

2.4. Hydraulic Modeling

2.4.1. 1D Model Calibration

The 1D hydraulic model calibration was conducted to ensure the channel's hydraulic capacity is realistically represented prior to overbank flow. Bankfull discharge was used as an indicator of the maximum channel capacity before flow overtops into the floodplain (Wohl, 2021). The calibration process involved evaluating peak discharge outputs from three synthetic unit hydrograph methods (SCS, Snyder, and Nakayasu) against the estimated bankfull capacity (Q2) of the Ciletuh River. Manning's roughness coefficients (n) were adjusted based on land use and river morphology to align the simulated water surface profiles with observed field conditions. This calibration focused on a specific observation point in the upstream section of the study area, where simulated overflow depths were compared against historical field records (50–60 cm). This comparative procedure is vital to identify the most suitable hydrological input for characterizing the watershed's hydraulic response within the HEC-RAS model.

2.4.2. Two-Dimensional (2D) Flood Inundation Simulation and Flow Velocity Simulation

Flood inundation was simulated using the two-dimensional (2D) module of HEC-RAS, which solves the shallow water equations using a finite volume approach to model the spatial distribution of flow depth and velocity (Brunner, 2024). The governing momentum equations solved in HEC-RAS 2D are derived from the depth-averaged Saint-Venant equations, which are expressed as follows:

$$\frac{\partial h}{\partial t} + \frac{\partial (hu)}{\partial x} + \frac{\partial (hv)}{\partial y} = 0 \quad (10)$$

where h is water depth (m), t is time (s), u is depth-averaged velocity component in the x-direction (m/s), v is depth-averaged velocity component in the y-direction (m/s), x is longitudinal spatial coordinate (m), and y is transverse spatial coordinate (m).

The model employed a mesh size of 30 m × 30 m to represent the study area topography efficiently. The upstream boundary condition was defined as the flood hydrograph derived from hydrological analysis, while the downstream boundary condition utilized a stage hydrograph based on mean water level elevations at the Ciletuh estuary, referring to observational data and sedimentation studies (Suryadi *et al.*, 2020).

The spatial validation procedure involved comparing the simulated Q2 inundation extent against historical flood records provided by the West Java Provincial Water Resources Agency (Dinas Sumber Daya Air Provinsi Jawa Barat). This process was designed to evaluate the model's performance in replicating reported annual inundation areas, specifically within agricultural land use. The validation analysis also accounted for potential discrepancies arising from Digital Elevation Model (DEM) resolution, local drainage conditions, and inherent uncertainties in field-based reporting. This systematic spatial verification was conducted to ensure the model's reliability in representing flood hazard characteristics across the downstream Ciletuh Watershed.

Table 2. Flood control scenarios

No	Scenario	Remarks
1	Existing	Baseline condition without structural intervention, used as the reference for evaluation.
2	Levee	Riverbank raising along segments prone to overtopping (Type I, II, and III).
3	Normalization	Channel deepening of 2–4 m and widening of 5–6 m along the study river segments.
4	Levee + Normalization	Combination of levee raising and enhancement of channel cross-sectional capacity.

2.4.3. Flood Control Scenarios

Flood control scenarios were formulated as structural interventions aimed at reducing inundation area and depth in the downstream segment of the Ciletuh Watershed, based on existing channel capacity, river morphological characteristics, and initial inundation simulation results. Four conditions were evaluated, as presented in Table 2: (1) existing condition,

(2) levee construction, (3) channel normalization, and (4) combined levee–normalization scenario. Each scenario was tested under 2-, 5-, 10-, and 25-year return period flood discharges to assess its effectiveness in reducing inundation extent and flood hazard classification.

2.4.3.1. Levee Scenario

The levee scenarios were developed to enhance the crest elevation of the riverbanks, establishing a structural barrier against overtopping during design flood events (Farid *et al.*, 2025). To ensure hydraulic consistency, all elevation data are strictly referenced to the Mean Sea Level (MSL) vertical datum (0.00 m), which serves as the fundamental benchmark for the hydraulic model. The levee types presented in Table 3 are defined by their relative crest elevation – representing a vertical increase above the existing bank level– to accommodate the natural upward slope of the Ciletuh River from downstream to upstream. The selection of these elevations was based on peak overtopping depths identified in base-line simulations, ensuring that the structural intervention effectively mitigates inundation hazards at critical segments.

Table 3. Levee flood control scenarios

Levee Type	Crest Elevation above Riverbanks (m)	Total Length (km)
Type I	+ 2.00	3.3
Type II	+ 1.50	2.3
Type III	+ 1.00	1.8

The spatial distribution of levee installation is shown in Figure 4a and was concentrated on segments exhibiting the highest inundation intensity and dominant overtopping points identified from the two-dimensional simulation results. This approach ensures that the structural intervention is aligned with the initial inundation pattern and effectively enhances the hydraulic capacity of the river channel.

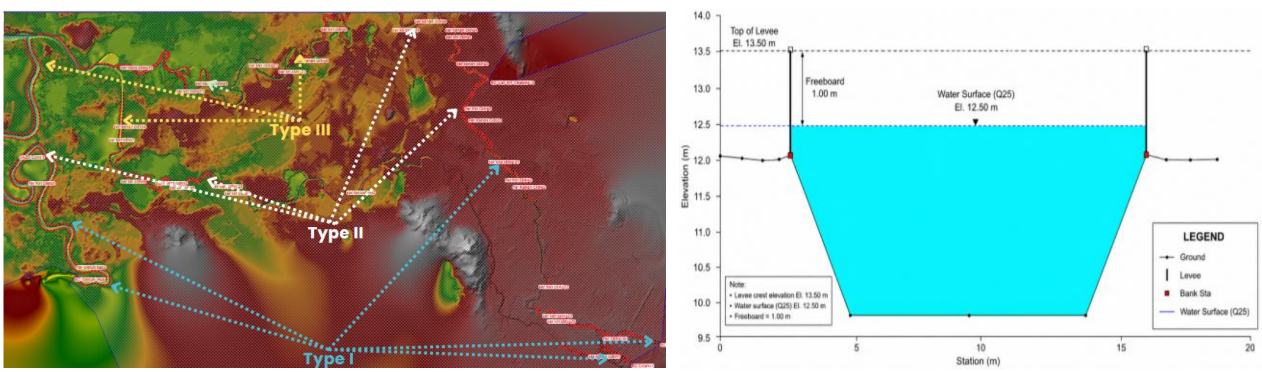


Figure 4. (a) Spatial distribution of type I, type II, and type III levee segments based on crest elevation classification in the downstream Ciletuh River, (b) River cross-section showing type II levee placement and the Q25 flood water level

The structural design of the levee is illustrated in Figure 4b, showing the typical geometry for a Type II levee. In this design stage, the crest elevation is defined by a 1.50-meter vertical offset above the existing riverbank to provide sufficient freeboard against the simulated Q25 flood water level.

2.4.3.2. Channel Normalization Scenario

The channel normalization scenario was implemented through riverbed deepening of 2–4 m and channel widening of 5–6 m along critical segments, as presented in Table 2. The geometric modifications were modeled in HEC-RAS 1D by adjusting the channel bed elevation and channel width parameters. The normalized channel was designed to maintain a consistent longitudinal bed slope between upstream and downstream sections, with average channel slope (S) of 0.002 to minimize excessive energy loss and maintain hydraulic flow continuity throughout the study reach.

A Manning's roughness coefficient (n) of 0.035 was applied to represent the normalized channel conditions, corresponding to a maintained earthen river channel with relatively uniform cross-sections and minimal vegetation cover. The selected value is consistent with standard hydraulic references for improved natural river reaches and was considered appropriate to represent flow resistance after channel widening and riverbed deepening. The normalization process was intended to restore the river conveyance capacity while maintaining stable hydraulic characteristics and consistent flow conditions along the downstream channel reach (Ahrendt *et al.*, 2022).

A comparison between the existing and normalized cross-sections at a representative segment is illustrated in Figure 5a. The increase in wetted cross-sectional area indicates an enhancement of flow conveyance capacity, which has the potential to reduce overbank flow into the floodplain.

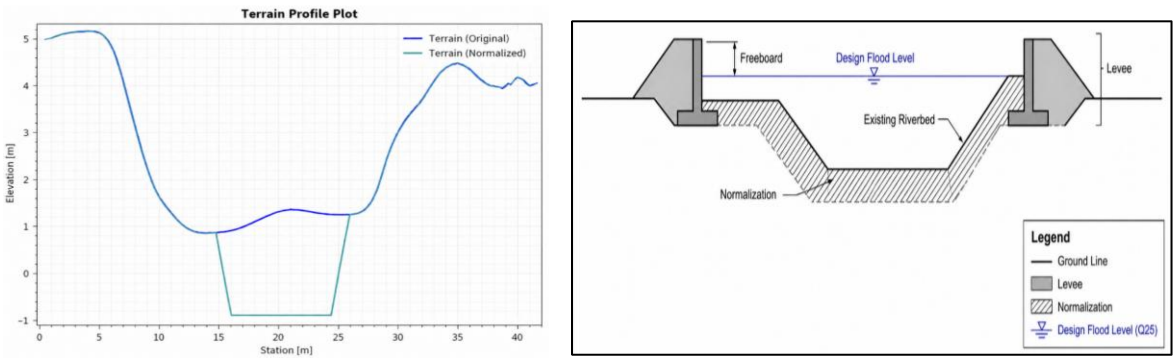


Figure 5. (a) River cross-section modification under the normalization scenario, (b) Cross-sectional configuration of the combined levee and channel normalization under Q25 flood conditions

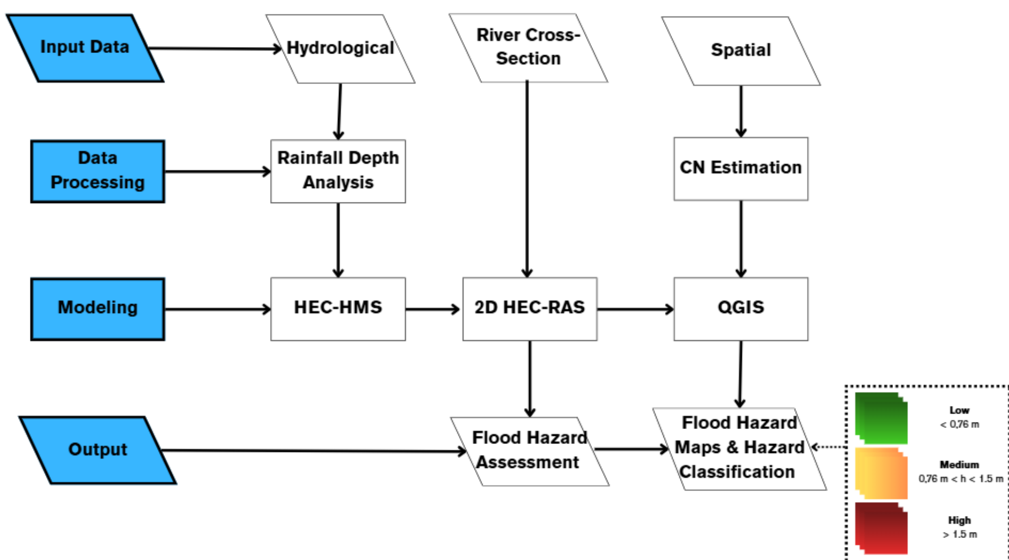


Figure 6. Research Flowchart

This approach enables a quantitative evaluation of the contribution of structural interventions in reducing flood hazard levels. The overall research framework is summarized in the flowchart presented in Figure 6.

2.4.3.3. Levee–Normalization Scenario

The combined scenario integrates channel capacity enhancement through normalization with increased protection elevation through levee construction along the downstream segment of the Ciletuh River. The modeling was conducted using HEC-RAS to evaluate, within a single simulation framework, the combined effects of increased flow conveyance capacity and overtopping control. Figure 5b illustrates the cross-sectional configuration of the combined levee and channel normalization scenario under Q25 flood conditions.

2.5. Hazard Index and Effectiveness Analysis

The inundation depths obtained from the two-dimensional simulations were subsequently analyzed to determine flood hazard levels within the study area. Hazard classification was conducted in accordance with the official guidelines established in the Regulation of the Head of the National Disaster Management Agency (BNPB) Number 2 of 2012 concerning the General Guidelines for Disaster Risk Assessment (BNPB, 2012). Inundation depth was used as the primary parameter, as it directly represents the potential for physical damage and the impact on community activities. Based on the inundation depth, flood hazard levels are categorized into three class, namely low (depth < 0.76 m), medium (1–1.5 m), and high (> 1.5 m) (BNPB, 2012). The effectiveness of flood control measures was calculated using the percentage reduction in inundation area as follows:

$$Reduction(\%) = \frac{A_{existing} - A_{scenario}}{A_{existing}} \times 100 \quad (11)$$

3. RESULTS AND DISCUSSION

3.1. Hydrological Analysis

Annual maximum rainfall data for the period 2004–2024, presented in Table 4, were obtained from two rainfall stations, namely Ciletuh and Maranginan. Prior to frequency analysis, the data were subjected to consistency and quality assessments. Data quality testing was conducted using outlier analysis and the Rescaled Adjusted Partial Sums (RAPS) test, as shown in Table 5, to ensure the absence of trend shifts or time-series inconsistencies.

Table 4. Annual maximum rainfall data

Year	Max Dialy Rainfall		Year	Max Dialy Rainfall		Year	Max Dialy Rainfall	
	Maranginan	Ciletuh		Maranginan	Ciletuh		Maranginan	Ciletuh
2004	105	97	2011	95	182	2018	85	185
2005	85	74	2012	61	150	2019	98	86
2006	69	89	2013	39	168	2020	102	208
2007	72	103	2014	96	132	2021	138	139
2008	76	180	2015	25	126	2022	86	137
2009	160	193	2016	24	158	2023	96	135
2010	110	112	2017	120	237	2024	92	167

Table 5. Outlier and RAPS test results

Rainfall Station	n	Dy (mm)	Q	R (mm)	Q/(n0.5)	R/(n0.5)	Result
Maranginan	20	33.1	2.2	4.11	0.49	0.92	Accepted
Ciletuh	20	42.36	2.16	3.85	0.47	0.84	Accepted

Note: n = number of data; Dy = cumulative deviation; Q = maximum cumulative deviation; R = range.

The outlier test results indicate that all data fall within the statistical control limits, with no extreme values requiring elimination. The RAPS test confirms that the data series is consistent and exhibits no significant trend changes during the observation period. Therefore, the dataset was deemed suitable for frequency analysis.

Rainfall frequency analysis was performed to estimate design rainfall for return periods of 2, 5, 10, and 25 years using the Normal, Log-Normal, Log-Pearson Type III, Gumbel, and Generalized Extreme Value (GEV) distributions, as presented in Table 6.

The selection of the best-fit distribution was carried out using the Kolmogorov–Smirnov test (Attained Smirnov), as summarized in Table 7. Based on the goodness-of-fit test, the GEV Max distribution was selected as it demonstrated the best agreement in representing the annual maximum rainfall data at both rainfall stations.

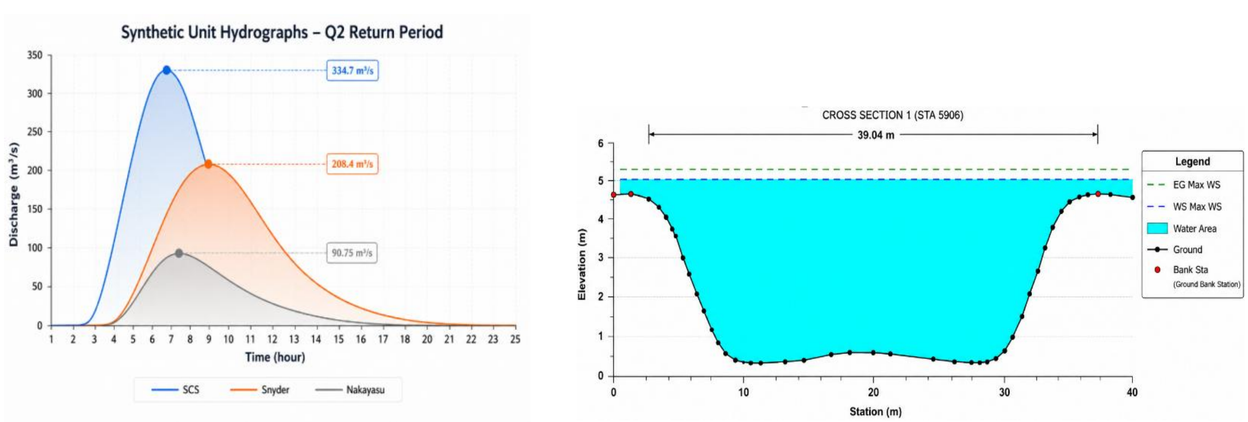
Table 6. Design rainfall estimates using various probability distributions

Distribution	Rainfall Station	Return Period			
		2	5	10	25
Normal	Maranginan	87.10	115.68	130.63	146.56
	Ciletuh	145.62	182.15	201.24	221.60
Log Normal	Maranginan	81.15	111.38	131.42	156.79
	Ciletuh	139.55	178.39	202.82	232.57
Log Pearson III	Maranginan	70.86	112.82	154.25	227.77
	Ciletuh	136.45	179.98	210.44	258.89
Gumbel	Maranginan	81.52	111.54	131.42	156.54
	Ciletuh	138.49	176.86	202.26	234.36
GEV Max	Maranginan	88.28	116.98	130.58	143.35
	Ciletuh	143.92	183.80	205.12	227.37

Table 7. Design rainfall (GEV Max)

Distribution	Rainfall Station	
	Maranginan	Ciletuh
Normal	97.4	100.0
Log Normal	52.5	96.9
Log Pearson III	9.6	96.3
Gumbel	55.8	93.1
GEV Max	99.3	100.0

Rainfall–runoff transformation was conducted using three synthetic unit hydrograph methods: Snyder, Nakayasu, and SCS, with the Q2 hydrographs presented in Figure 7(a). The SCS Unit Hydrograph method refers to the unit hydrograph concept developed in the National Engineering Handbook (USDA-SCS, 1972). The selection of the most representative method was performed through calibration against the bankfull discharge, which was approximated by the 1.5–2-year return period discharge (Q2), as it hydrologically represents the channel capacity before overbank flow occurs.



(a) (b)

Figure 7. (a) Q2 hydrograph and (b) Bankfull overtopping under Q2 discharge

Based on calibration results against Q2, the SCS method exhibited the closest agreement with the watershed response characteristics, as summarized in Table 8. The comparison indicates that the Snyder and Nakayasu methods produced peak discharges lower than the estimated bankfull capacity, whereas the SCS Unit Hydrograph generated a Q2 discharge that most closely matched the observed hydraulic condition. Therefore, the SCS method was selected for one-dimensional (1D) hydraulic modeling using HEC-RAS, as illustrated in Figure 7(b). The observation point used for calibration represents a frequently inundated area located in the upstream section of the study area, which is still situated within the downstream part of the Ciletuh Watershed. The simulation results indicate overflow depths consistent with historical field observations, where annual inundation depths generally range between 50–60 cm.

Table 8. Comparison of Q2 peak discharge results using different unit hydrograph methods

Method	Q2 Peak Discharge (m^3/s)	Difference to Bankfull	Calibration Result
SCS	334.70	Closet to Observed Overflow	Selected
Snyder	208.40	Underestimated	Less Representative
Nakayasu	90.75	Strongly Underestimated	Poor Fit

Furthermore, two-dimensional (2D) model validation was conducted by comparing the simulated inundation extent under the Q2 scenario with historical flood records reported by the West Java Provincial Water Resources Agency, as presented in Figure 8. The agency documented that annual flooding in the downstream Ciletuh Watershed regularly inundates approximately 200 ha of rice field areas, resulting in recurring agricultural losses each year. In comparison, the HEC-RAS 2D simulation produced an inundated agricultural area of 246.61 ha. Validation through comparison between simulated and observed flood inundation is swidely recognized as an essential procedure for evaluating the reliability of hydraulic flood models and flood hazard mapping (Thapa *et al.*, 2020; Tamiru & Dinka, 2021). The relatively close agreement between reported and simulated inundation extents indicates that the model reasonably represents the spatial flood distribution and inundation characteristics within the study area. Minor discrepancies may be influenced by DEM resolution, local drainage conditions, and uncertainties in field-based inundation reporting.

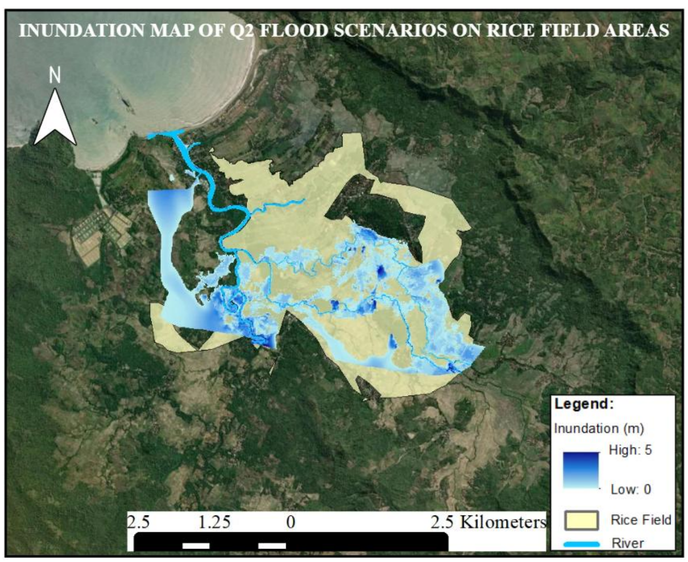


Figure 8. Inundtaion map of Q2 flood scenarios on rice field areas in Ciemas Disctrict, Ciletuh Watershed

The hydrological response of the Ciletuh Watershed to the design rainfall was further analyzed using the selected synthetic unit hydrograph, as illustrated in Figure 9. The simulation results reveal a responsive watershed characterized by a sharp rise in discharge and a relatively short time to peak. Peak discharge increases significantly with increasing return period, from 352.6 m^3/s (Q2) to 716.3 m^3/s (Q25). The time to peak occurs between the 7th and 8th hour, reflecting a relatively short concentration time influenced by steep upstream topography. The steep recession limb indicates limited watershed storage capacity, thereby increasing flood potential in the downstream area.

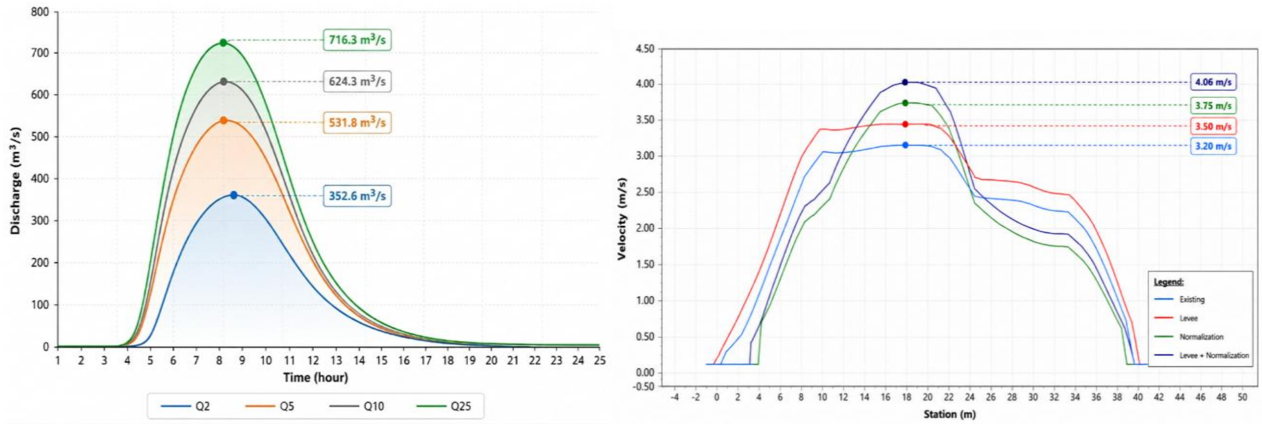


Figure 9. (a) Design flood hydrographs for different return period (Tr), and (b) Flow velocity distribution under different flood control scenarios

3.2. Hydraulic Analysis

3.2.1. Flow Velocity Analysis

The HEC-RAS 2D simulation results indicate that flood control measures significantly influenced the flow velocity characteristics in the downstream section of the Ciletuh Watershed. Figure 13 shows the velocity distribution under the existing, levee, normalization, and combined levee–normalization scenarios. The combined scenario produced the highest peak velocity of approximately 4.06 m/s, followed by the normalization scenario (3.75 m/s), levee scenario (3.50 m/s), and existing condition (3.20 m/s).

The increase in velocity under the modified scenarios reflects improved hydraulic conveyance resulting from channel enlargement and flow confinement within the river section. Hydraulically, channel normalization reduces flow resistance, while the levee system limits lateral flow spreading and concentrates discharge within the main channel. Previous studies have shown that structural modifications of river channels can increase flow concentration and hydraulic conveyance while also creating localized scour in areas subjected to higher flow velocities (Czapiga *et al*., 2022). Although these interventions improve flow efficiency during flood events, the higher velocity may also increase the potential for local scour and bank erosion in downstream areas.

3.2.2. Flood Inundation Analysis

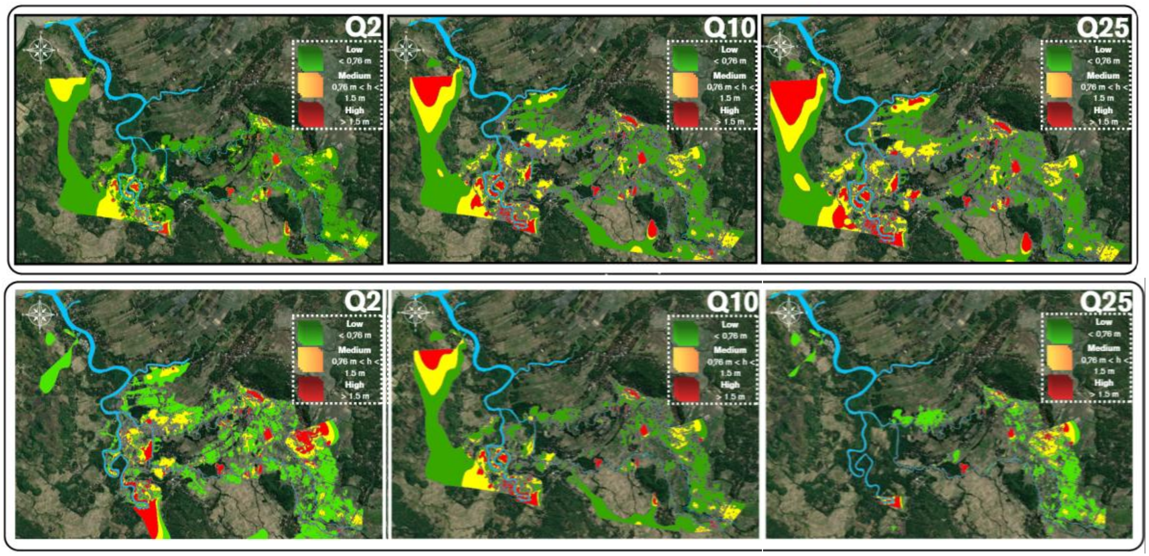
a. Existing Condition

A summary of inundation area and depth classification is provided in Table 10. For the 2-year return period (Q2), the inundated area reaches 358 ha, predominantly within the depth class of <0.76 m (279 ha). However, under the 25-year return period (Q25), the inundation area increases substantially to 584 ha. The high-depth class (>1.5 m) expands markedly from 16 ha (Q2) to 77 ha (Q25), indicating a considerable escalation in flood hazard level.

The two-dimensional simulation results indicate that both inundation extent and flood depth increase significantly with higher return periods. The spatial distribution of the flood hazard index under existing conditions (Figure 10top) illustrates the expansion of inundated areas, particularly along the downstream segment and adjacent floodplains.

Table 10. Recapitulation of inundation area and depth classes under existing conditions

Tr	Inundation (ha) <0.76m	0.76-1.5m	>1.5m	Total	Residential (unit)	Rice Field (ha)	Agriculture (ha)
Q2	279	63	16	358	154	247	67
Q5	333	115	38	486	393	342	88
Q10	353	137	54	544	136	386	99
Q25	350	156	77	584	121	413	108



(a) Levee (b) Normalization (c) Levee + Normalization

Figure 10. Spatial distribution and comparison of flood hazard index (FHI) across various return periods under: (top) Existing conditions, (bottom) Flood control scenarios

In addition to the increase in inundation extent, land-use impacts also intensify. Under Q25 conditions, flooding affects approximately 413 ha of paddy fields and 108 ha of other agricultural land. These findings confirm the high vulnerability of the downstream Ciemas District to extreme flood events, particularly in productive agricultural areas.

b. Flood Control Scenarios

The spatial distribution of the flood hazard index for the 25-year return period (Q25) under various flood control scenarios is shown in Figure 10bottom. Structural interventions clearly reduce both inundation extent and high-depth classes compared to existing conditions.

Table 11 summarizes the inundation areas for all scenarios and return periods. Under Q25 conditions, the levee scenario reduces the inundated area from 584 ha to 431 ha. The channel normalization scenario performs better, decreasing the inundation area to 399 ha. However, the combined levee–normalization scenario provides the most substantial improvement, reducing the inundation area to only 152 ha. This reduction is observed not only in total inundation area but also in the high-depth class (>1.5 m). Under the combined Q25 scenario, the high-depth inundation area decreases dramatically to 8 ha compared to 77 ha under existing conditions. These results demonstrate that the integrated structural approach is more effective in reducing flood hazard levels.

Table 11. Recapitulation of inundation area and depth classes under flood control scenarios

Scenario	Tr	Inundation (ha)				Residential (unit)	Rice Field (ha)	Agriculture (ha)
		<0.76m	0.76-1.5m	>1.5m	Total			
Levee	Q2	107	22	6	136	63	132	3
	Q5	207	46	13	267	195	235	7
	Q10	253	67	25	345	244	301	10
	Q25	295	91	45	431	265	364	15
Normalization	Q2	88	17	3	108	18	91	16
	Q5	211	61	13	285	111	166	70
	Q10	331	12	0	343	118	211	78
	Q25	361	38	0	399	173	257	86
Levee + Normalization	Q2	52	5	1	58	14	57	1
	Q5	82	20	2	104	32	98	2
	Q10	93	25	5	123	47	112	6
	Q25	112	32	8	152	62	136	7

c. Inundation Reduction Analysis

The percentage reduction in inundation area relative to existing conditions for Q25 is summarized in Table 12. The levee scenario achieves a reduction of 26%, while channel normalization results in a 32% reduction. The combined levee–normalization scenario demonstrates the highest effectiveness, achieving a 74% reduction in inundation area.

The visual comparison of Q25 inundation reduction between the existing condition and the flood control scenarios is presented in Figure 11. The graph indicates a substantial decline in total inundation area, along with a redistribution of flood depth classes toward predominantly low and moderate categories under the combined scenario, reflecting a marked reduction in overall flood hazard intensity.

Table 12. Percentage reduction of inundation area compared to existing conditions (Q25)

Scenario	Total Inundation (ha)	Reduction (%)
Existing	584	-
Levee	431	26%
Normalization	399	32%
Levee + Normalization	152	74%

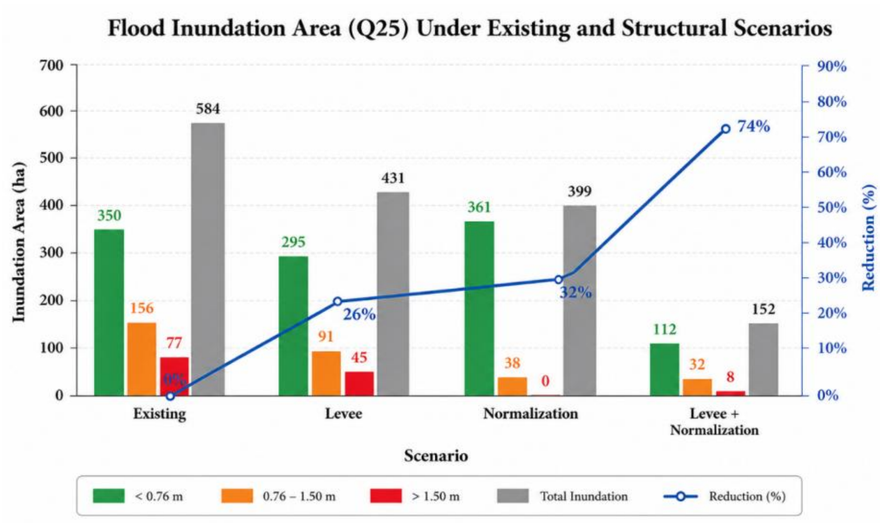


Figure 16. Comparison of Q25 inundation area between existing conditions and flood control scenarios

From a hydraulic perspective, the effectiveness of the combined scenario is attributed to the simultaneous increase in channel conveyance capacity through normalization, which reduces water surface elevation, and the enhancement of overflow protection through levee construction, which prevents lateral spill into the floodplain. The synergy of these structural measures results in a substantial reduction of inundation extent in the downstream Ciletuh Watershed.

4. CONCLUSIONS

The results of hydrological simulation and two-dimensional hydraulic modeling in the downstream Ciletuh Watershed demonstrate that increasing return periods are directly proportional to increases in inundation extent and flood depth. Under the existing Q25 condition, the inundation area reaches 584 ha, with a significant expansion in the high-depth class (>1.5 m), predominantly affecting productive paddy fields and posing considerable flood hazard impacts. The evaluated structural control scenarios exhibit varying levels of effectiveness. The combined levee–normalization scenario provides the most substantial improvement, with a 74% reduction in inundation area; however, this value is subject to a certain degree of uncertainty due to methodological limitations. This enhanced performance is attributed to the simultaneous increase in channel capacity and protection elevation. In terms of hydraulic response, the modified scenarios also generated higher flow velocities compared to the existing condition, with the combined levee–

normalization scenario producing the highest peak velocity. The increase in velocity indicates improved hydraulic conveyance efficiency due to channel enlargement and flow confinement within the river section, although it may also increase the potential for local scour and bank erosion in downstream areas. Implementation of the combined levee–normalization scenario should be further assessed through detailed technical and economic feasibility analyses. This study has integrated the BNPB flood hazard index framework to classify and map flood-prone areas, providing a robust baseline for flood mitigation planning in the Ciletuh River. Future research is recommended to incorporate climate change projections to further refine these mitigation strategies.

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DECLARATION OF GENERATIVE AI AND AI-ASSISTED TECHNOLOGIES IN THE MANUSCRIPT PREPARATION

During the preparation of this work, the authors used Gemini and ChatGPT in order to develop the conceptual framework, identify relevant theoretical foundations, and gather preliminary references related to this research. After using these tools/services, the authors reviewed and edited the content as needed and take full responsibility for the content of the publication.

AUTHOR CONTRIBUTION

Author	C	M	So	Va	Fo	I	R	D	O	E	Vi	Su	P	Fu
MPS	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓			✓
JN	✓	✓		✓	✓					✓		✓		
WD	✓	✓		✓	✓					✓		✓		
ANC										✓		✓		
RMJ									✓	✓	✓		✓	

C: Conceptualization	Fo: Formal Analysis	O: Writing - Original Draft	Fu: Funding Acquisition
M: Methodology	I: Investigation	E: Writing - Review & Editing	P: Project Administration
So: Software	D: Data Curation	Vi: Visualization	
Va: Validation	R: Resources	Su: Supervision	

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