



Rainfall validation, runoff estimation, and drainage capacity assessment for a residential development

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Received Date: April 29, 2026

Revised Date: August 18, 2026

Accepted Date: August 24, 2026

ABSTRACT

Background: Rapid conversion of peri-urban land to housing increases impervious cover, accelerates runoff, and can overload small drainage corridors. This study evaluated the hydrological baseline and proposed stormwater infrastructure for the Sunville Residence development in Paccinongang, Gowa Regency, Indonesia.

Methods: The assessment integrated creek geometry and velocity surveys conducted in June and December 2025, NASA Global Precipitation Measurement rainfall for 2015–2024, a 24-month comparison with nearby Meteorology, Climatology, and Geophysics Agency gauge maxima, partial-duration screening above 50 mm/day, reported Gumbel design rainfall, Mononobe rainfall intensity, the Rational Method, open-channel hydraulic checks, detention-volume calculations, and suspended-sediment loads. Satellite validation used bias, mean absolute error, root mean square error, Pearson correlation, percent bias, and Nash–Sutcliffe efficiency.

Findings: Observed creek discharge ranged from 0.495 to 0.660 m³/s. NASA rainfall underestimated gauge maxima by 8.98 mm on average, with a mean absolute error of 18.13 mm, root mean square error of 24.75 mm, correlation of 0.68, percent bias of –19.82%, and Nash–Sutcliffe efficiency of 0.36. The reported 10-year design rainfall and intensity were 122.91 mm and 7.34 mm/h. Estimated runoff increased from 1.29 m³/s during construction to 1.80 m³/s during operation as the runoff coefficient rose from 0.25 to 0.35. The proposed channel capacity was 1.90 m³/s, leaving only 0.10 m³/s or 5.6% hydraulic margin. An illustrative aggregate bias adjustment increased operational runoff to 2.24 m³/s, above the stated capacity. A 13,000 m³ sedimentation pond marginally exceeded the corrected 8-hour storage requirement of 12,960 m³. Estimated sediment load increased from 1.25 t/day upstream to 1.40 t/day downstream. **Conclusion:** The proposed system is nominally adequate under the reported design event, but its narrow capacity and storage margins require reconciliation of input inconsistencies, sensitivity testing, clogging allowances, and integrated source-control measures before final engineering approval. **Novelty/Originality of this article:** The study combines hydrological calculation with a reproducibility audit that identifies numerical and documentation inconsistencies that materially affect drainage safety decisions in a fast-growing Indonesian residential catchment.

KEYWORDS: drainage design; global precipitation measurement; rainfall validation; rainfall–runoff modelling; reproducibility audit.

1. Introduction

Urban and peri-urban residential development modifies the hydrological response of land by replacing vegetated and agricultural surfaces with roofs, roads, compacted soil, and

Cite This Article:

Syahrir, M., Dewi, C., & Daud, A. M. (2026). Rainfall validation, runoff estimation, and drainage capacity assessment for a residential development. *EcoVision: Journal of Environmental Solutions*, 3(2), 325–346. <https://doi.org/10.61511/evojes.v3i2.2026.3933>

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other relatively impervious materials. These changes reduce infiltration and surface storage, shorten travel time, and increase the volume and peak rate of stormwater delivered to drainage networks (Shuster et al., 2005; Jacobson, 2011; Miller et al., 2014). Conventional systems that focus only on rapid conveyance can transfer flood risk downstream and intensify erosion or receiving-water impacts. Contemporary urban drainage therefore combines reliable conveyance with detention, infiltration, sediment control, maintenance access, and preservation of natural flow paths (Burns et al., 2012; Fletcher et al., 2015; Eckart et al., 2017; Zhou, 2014; Rossman & Huber, 2016).

Small natural or semi-engineered creeks are particularly vulnerable when a housing project occupies part of their catchment. Such channels often receive runoff from areas outside the project boundary, yet design calculations may be based primarily on the development parcel. A drainage system can appear adequate when evaluated against local site runoff but fail if upstream contributing areas, backwater, debris, sediment accumulation, or coincident flows are not represented. Modern drainage guidance consequently requires explicit definition of the contributing catchment, design frequency, time of concentration, rainfall intensity, roughness, tailwater, freeboard, and maintenance assumptions (Federal Highway Administration, 2024).

Indonesian drainage design is governed by a layered national framework linking plot-scale rainwater management, urban drainage planning, and design-flood estimation. Regulation of the Minister of Public Works No. 11/PRT/M/2014 addresses rainwater management for buildings and their plots, while Regulation No. 12/PRT/M/2014 governs implementation of urban drainage systems. For hydrological design, SNI 2415:2026, *tata cara perhitungan debit banjir rencana* (procedures for calculating planned flood discharge), is the current national standard for design-flood calculation; the 2026 revision adds rainfall-runoff modelling and updated calculation guidance. For a residential development discharging to a shared creek, these instruments reinforce the need to document the relationship between plot-scale controls and catchment-scale conveyance, including design frequency, contributing area, rainfall input, runoff transformation, storage, downstream boundary conditions, and maintenance assumptions.

Frequency analysis should therefore be treated as a model-selection problem rather than the routine adoption of a single default distribution. The Gumbel or extreme value type I distribution is widely used for hydrological extremes, but its tail form is fixed (Gumbel, 1958). The generalized extreme value (GEV) family introduces a shape parameter and can represent a broader range of upper-tail behaviour, whereas Log-Pearson type III (LP3) accommodates skewness after logarithmic transformation and is the recommended framework for annual peak-flow frequency analysis in U.S. Bulletin 17C (Coles, 2001; England et al., 2018). Where a partial-duration or peaks-over-threshold series is used, generalized Pareto threshold models can retain more extreme-event information than annual maxima, but only when the threshold is defensible and exceedances are sufficiently independent or declustered (Coles, 2001). Regional frequency analysis based on L-moments offers another option for short at-site records by pooling information from hydrologically homogeneous sites (Hosking & Wallis, 1997). Indonesian practice likewise demonstrates the value of comparison rather than automatic selection: A 2025 Batam drainage study evaluated Normal, Log-Normal, Gumbel, and Log-Pearson type III distributions and selected LP3 using Kolmogorov-Smirnov and Anderson-Darling goodness-of-fit diagnostics (Rahayu et al., 2025). Accordingly, distribution choice should be justified using goodness-of-fit diagnostics, tail plausibility, sample length, data structure, and uncertainty.

These catchment-scale effects also have a social dimension. When stormwater from a new development is conveyed rapidly into a shared creek or canal, hydraulic risk can be transferred beyond the project boundary rather than eliminated, with possible consequences for downstream flooding, erosion, sediment deposition, drainage access, and receiving-water conditions. From an environmental-justice perspective, the relevant question is therefore not only whether the project drainage system passes a nominal hydraulic check, but also who bears the residual risk and maintenance burden when runoff is transferred downstream. The present source material does not contain demographic or

household-level vulnerability data, so this study does not claim that particular communities are disproportionately affected. Instead, it identifies downstream risk distribution as a necessary component of socially responsible and sustainable drainage assessment (Burns et al., 2012; Fletcher et al., 2015; Zhou, 2014).

Rainfall data are the first major source of uncertainty. Gauge observations provide direct local measurement but may be short, incomplete, or spatially sparse; global gauge coverage remains highly uneven (Kidd et al., 2017). Satellite precipitation products provide consistent spatial coverage and are valuable in data-limited regions. The Global Precipitation Measurement (GPM) mission and its integrated multi-satellite retrievals for GPM products combine passive microwave, infrared, and gauge information to produce global precipitation estimates (Hou et al., 2014; Tan et al., 2017). Their performance varies with temporal scale, rainfall regime, season, topography, and storm intensity. Studies over the Indonesian maritime continent have reported useful daily detection but also non-negligible bias and event-scale error, which makes local validation indispensable before hydrological design use (Ramadhan et al., 2022).

Validation should not rely on a single metric. Mean bias indicates systematic overestimation or underestimation, mean absolute error describes average error magnitude, and root mean square error gives greater weight to large deviations. Correlation measures co-variation but does not establish agreement, while Nash–Sutcliffe efficiency compares residual variance with observed variance. Percent bias expresses volumetric deviation. Joint interpretation is important because a product may track seasonal patterns yet underestimate extreme events, which are precisely the events that dominate drainage design (Moriassi et al., 2007; Kling et al., 2012).

Peak-flow estimation for small catchments often uses the rational method because it is transparent and requires a limited number of inputs. The method relates peak discharge to runoff coefficient, design rainfall intensity, and contributing area. Its simplicity is useful for preliminary design, but its assumptions become restrictive when catchments are heterogeneous, storage is substantial, rainfall is spatially variable, or routing effects matter. The selected runoff coefficient embeds land cover, soil, slope, antecedent condition, and drainage connectivity. The rainfall intensity must correspond to a duration at least equal to the catchment time of concentration. For final design, the calculation should be checked with sensitivity analysis or a dynamic model where data permit (Chow et al., 1988; Akan & Houghtalen, 2003; Rossman & Simon, 2022).

Drainage adequacy also depends on hydraulic rather than hydrological capacity alone. Manning-based open-channel design requires channel geometry, longitudinal slope, roughness, flow depth, and permissible velocity. A calculated capacity slightly above the design discharge may be inadequate in practice because vegetation, sediment, debris, construction tolerances, inlet losses, downstream water levels, and aging reduce effective capacity. Freeboard provides a geometric allowance, but it does not replace a quantified reliability margin or maintenance plan (Federal Highway Administration, 2024).

Sediment management is equally important during land preparation. Earthworks increase soil exposure, and suspended solids can accumulate along channels and in receiving waters. Detention or sedimentation ponds reduce peak flow and promote settling, but performance depends on storage volume, detention time, hydraulic short-circuiting, inlet energy dissipation, outlet configuration, and the volume reserved for deposited sediment. Stormwater ponds can retain particulate contaminants, yet accumulated sediment ultimately requires inspection and removal (Flanagan et al., 2021; Maniquiz-Redillas et al., 2022). A pond sized with almost no reserve above its nominal design volume is operationally fragile.

The proposed Sunville Residence in Paccinongang, Gowa Regency, is intersected by a creek that conveys runoff northward toward the Tamangapa Canal. The existing project hydrology document reported field measurements, satellite rainfall, gauge validation, extreme-rainfall calculations, runoff estimates, a trapezoidal channel design, a sedimentation pond, and sediment loads. However, the material was structured as an environmental baseline chapter rather than a reproducible scientific analysis. Several

internal inconsistencies were also present, including conflicting data-period labels, an incorrect conversion of eight hours to seconds, limited reporting of hydraulic inputs, and a peak-flow value that was not reconciled with the subsequent rational method calculation (PT Bumi Surya Persada, 2025). These characteristics motivate an audit question: whether the existing evidence chain can be independently reconstructed well enough to support the stated drainage-safety conclusion.

Accordingly, this manuscript is framed as a secondary analysis and reproducibility audit of the project hydrology package, rather than as a new field-based hydrological investigation. The existing creek observations and rainfall records are used as auditable source data rather than presented as newly generated field evidence. The study addresses one focused research question: To what extent can the reported hydrological design chain—from rainfall inputs and runoff estimation to channel and detention-pond adequacy—be independently reproduced from the available project records, and do missing inputs or internal inconsistencies materially affect the interpretation of drainage safety?

2. Methods

This study used a secondary-analysis design. All hydrological observations and design inputs were drawn from the existing project hydrology report and associated records; no additional field campaign was undertaken specifically for this manuscript. The analytical work consisted of recomputation, cross-checking, uncertainty interpretation, illustrative sensitivity testing, and assessment of whether the reported design chain was sufficiently documented for independent reproduction. The sensitivity component was used as a diagnostic stress test of decision-sensitive assumptions, not as a fully calibrated or independently validated hydrological-hydraulic model.

2.1 Study area and hydrological setting

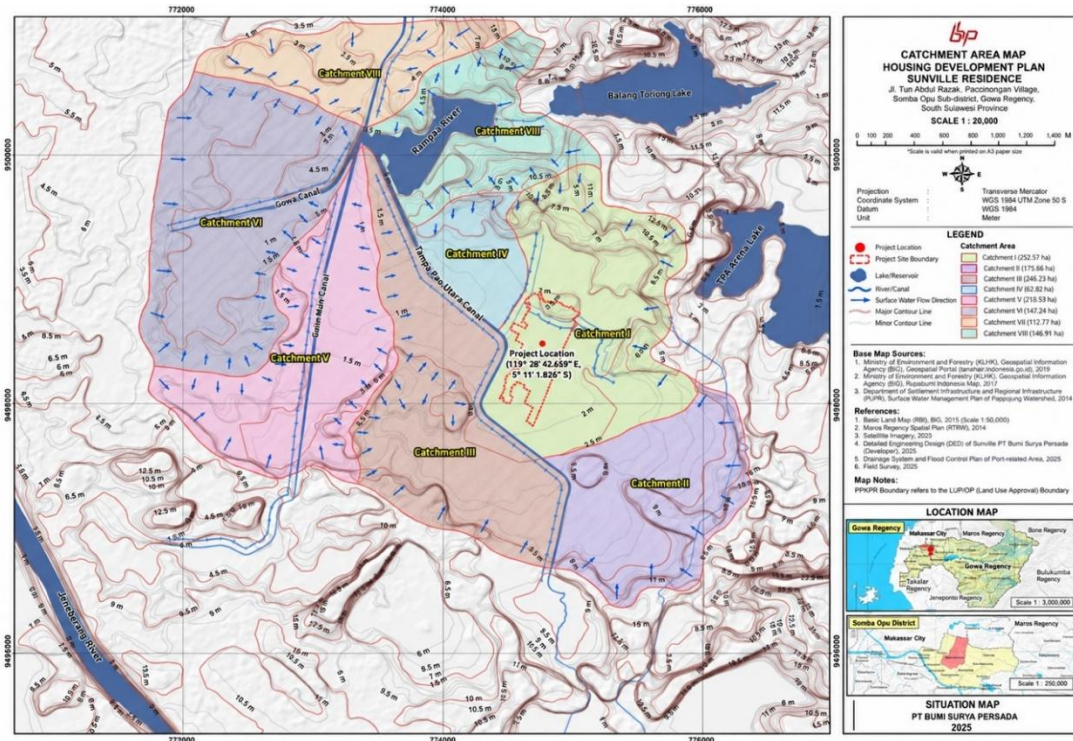


Fig. 1. Catchment delineation and project location in Paccinongang, Gowa Regency

The study concerned the proposed Sunville Residence development in Paccinongang Village, Somba Opu District, Gowa Regency, South Sulawesi, Indonesia. A small creek crosses or borders the northern part of the project area and drains toward the Tamangapa Canal.

The mapped catchment extends beyond the housing parcel toward areas north and east of the site. This configuration means that stormwater management must account for external inflow as well as runoff generated by the development itself. The catchment map prepared for the project divides the surrounding terrain into several contributing subareas and shows convergent flow toward the creek and canal.

2.2 Creek survey and field discharge

The source project report documents creek surveys conducted in June 2025 and December 2025. Channel width and depth were reported at upstream, middle, and downstream sections where accessible, and flow velocity was recorded in metres per second. The project report calculated instantaneous discharge as the product of wetted cross-sectional area and mean velocity, $Q = A \times v$, and recorded substrate condition visually. For this manuscript, these measurements were re-used as source-project observations and their arithmetic and interpretive limits were audited. Because the source document reported rectangular area estimates and did not provide multiple depth verticals, velocity profiles, measurement duration, or instrument calibration, the results were treated as reconnaissance discharges rather than full hydrometric ratings.

2.3 Rainfall data and satellite validation

The hydrological analysis used NASA Global Precipitation Measurement (NASA GPM) rainfall for 2015–2024. The source dataset consisted of monthly maximum daily rainfall values. Heavy-rainfall observations exceeding 50 mm/day were selected as a partial-duration series. This threshold was retained from the project document. Independence between exceedances was not documented, so the series was interpreted as a screening sample rather than a formally declustered peaks-over-threshold dataset.

Satellite rainfall was compared with 24 paired monthly maxima from a nearby Meteorology, Climatology, and Geophysics Agency station. The numerical table is dated January–December 2023 and January–December 2024, whereas the accompanying source narrative refers to 2024–2025. Because each numerical pair is explicitly attached to the tabulated dates, this audit adopts 2023–2024 as the operative gauge–satellite validation period. The unsupported 2024–2025 narrative label is treated as a source-document labeling inconsistency and is not used in any calculation unless raw dated records are later provided to substantiate it. Validation metrics were calculated as follows: mean bias, $B = n^{-1}\Sigma(S_i - O_i)$; mean absolute error (MAE) = $n^{-1}\Sigma|S_i - O_i|$; root mean square error (RMSE) = $[n^{-1}\Sigma(S_i - O_i)^2]^{1/2}$; Pearson correlation, r ; percent bias (PBIAS) = $100\Sigma(S_i - O_i)/\Sigma O_i$; and Nash–Sutcliffe efficiency (NSE) = $1 - \Sigma(S_i - O_i)^2/\Sigma(O_i - \bar{O})^2$. Here S is the satellite estimate and O is the gauge observation.

2.4 Design rainfall and runoff

The project report applied Gumbel frequency analysis to the selected rainfall data and reported design rainfall for return periods from 2 to 25 years. For reproducibility, the reported return-period curve was reconstructed with the standard Gumbel reduced variate $y(T) = -\ln[-\ln(1 - 1/T)]$ and $X(T) = \mu + \beta y(T)$. Fitting the six reported return-period values gives an equivalent location parameter $\mu = 29.47$ mm and scale parameter $\beta = 41.52$ mm; these parameters reproduce the published design-rainfall values to rounding precision. They are reconstructed equivalent parameters and should not be interpreted as the original, undocumented sample statistics used by the project consultant. Rainfall intensity was reported as derived using a Mononobe-type relationship, $I = (R24/24)(24/tc)^{(2/3)}$. Inverting the reported 10-year values ($R24 = 122.91$ mm and $I = 7.34$ mm/h) gives an implied effective duration $tc = 13.99$ h (839 min). The source records available for this audit do not provide the flow-path length L , elevation difference, or catchment slope S required to reproduce the Kirpich (1940) calculation, $tc = 0.01947L^{0.77}S^{-0.385}$ (min, with L in m

and S in m/m). Accordingly, the 13.99 h value is reported as the duration implied by the published Mononobe intensity, not as an independently verified Kirpich result. The 10-year event was retained for the subsequent runoff design.

Peak runoff was calculated using the metric rational method, $Q = 0.278CIA$, where Q is peak discharge in m^3/s , C is the dimensionless runoff coefficient, I is rainfall intensity in mm/h, and A is catchment area in km^2 . A catchment area of 2.52 km^2 was used. Construction conditions used $C = 0.25$, while operation used $C = 0.35$ to represent increased imperviousness and connectivity. The increase was evaluated both in absolute discharge and percentage terms.

2.5 Channel, pond, and sediment calculations

The reported trapezoidal-channel design was assessed by comparing its stated hydraulic capacity with the operational design discharge. Capacity margin was calculated as $Q_{\text{cap}} - Q_{\text{design}}$ and as a percentage of Q_{design} . The source document supplied cross-sectional geometry, mean velocity, hydraulic radius, and bed slope but did not state Manning roughness. For audit purposes, Manning n was back-calculated from the reported mean velocity using $n = R^{(2/3)}S^{(1/2)}/v$, with $R = 0.45 \text{ m}$, $S = 0.005$, and $v = 1.38 \text{ m/s}$, giving $n = 0.030$. An independent back-calculation using the stated capacity ($1.90 \text{ m}^3/\text{s}$) and the trapezoidal flow area $A = 1.446 \text{ m}^2$ gives $v = 1.314 \text{ m/s}$ and an equivalent $n = 0.032$. The small difference confirms that the reported velocity, geometry, and capacity are not perfectly self-consistent; therefore, $n \approx 0.030$ is used only as a reconstructed audit value and not presented as a documented original design input.

The sedimentation pond was evaluated using $V = Qt$. The source stated $Q = 0.45 \text{ m}^3/\text{s}$, an eight-hour detention period, and a required volume of $12,960 \text{ m}^3$. Eight hours equals 28,800 seconds, not 43,200 seconds as written in one line of the source document. The corrected multiplication, $0.45 \times 28,800$, confirms the reported required volume. The proposed dimensions were $100 \text{ m} \times 52 \text{ m} \times 2.5 \text{ m}$, corresponding to $13,000 \text{ m}^3$ before allowances for side slopes, dead storage, sediment reserve, or freeboard.

Suspended-sediment load was checked from discharge and TSS concentration using $L = 0.0864QC$, where L is tonnes per day, Q is m^3/s , and C is mg/L. Values were calculated for upstream, middle, and downstream sections of the Tamangapa Canal. No particle-size distribution or settling-velocity data were available, so the pond assessment addresses hydraulic storage rather than sediment-removal efficiency.

2.6 Analytical quality control and reproducibility audit

The reproducibility audit was the primary analytical procedure of the manuscript. All tabulated calculations were recomputed from the reported inputs, and the calculation chain was traced from rainfall data and frequency analysis through runoff estimation, channel capacity, detention storage, and sediment load. Apparent typographical, dimensional, chronological, and cross-table inconsistencies were retained as documented observations rather than silently removed. The audit distinguished among; (1) results that could be reproduced directly from reported inputs; (2) errors that could be corrected from unambiguous information, such as the eight-hour conversion; and (3) omissions that prevented independent reconstruction, such as the missing Manning coefficient and time-of-concentration inputs. The sensitivity calculations were deliberately illustrative deterministic perturbations used only to test whether plausible changes in uncertain inputs could alter the engineering pass-fail interpretation. They should not be interpreted as calibrated or fully validated hydrological or hydraulic model outputs, forecasts of future conditions, or estimates of failure probability. This approach follows reproducibility principles requiring assumptions, provenance, intermediate values, and decision thresholds to be visible to reviewers and decision-makers.

3. Results and Discussion

3.1 Creek morphology and observed flow

The creek had a narrow, shallow-to-moderate cross-section with widths of 1.50–1.85 m and reported depths of 1.25–1.50 m. The substrate consisted of sand and silt, and field photographs show vegetated earth banks in some reaches and a lined channel in another. Such variability can create local controls, backwater, and sediment trapping. Vegetation may reduce erosion but can also increase hydraulic resistance when unmanaged.

Table 1. Field morphology and instantaneous discharge of the project creek

Survey	Reach	Width (m)	Depth (m)	Velocity (m/s)	Area (m ²)	Q (m ³ /s)	Bed
June 2025	Upstream	1.50	1.50	0.22	2.25	0.495	Sand and silt
June 2025	Middle	1.50	1.50	0.24	2.25	0.529	Sand and silt
June 2025	Downstream	1.85	1.25	0.25	2.31	0.578	Sand and silt
December 2025	Upstream	1.50	1.50	0.26	2.25	0.580	Sand and silt
December 2025	Downstream	1.50	1.50	0.29	2.25	0.660	Sand and silt

Instantaneous discharge increased from 0.495 m³/s upstream to 0.578 m³/s downstream in June. December measurements were higher, reaching 0.580 m³/s upstream and 0.660 m³/s downstream. The pattern is consistent with wetter conditions and cumulative inflow along the channel, although two seasonal surveys are insufficient to define a rating curve or flood discharge. The middle June discharge in the source table was 0.529 m³/s, whereas direct multiplication of 2.25 m² by 0.24 m/s gives 0.540 m³/s. This 2.1% discrepancy is small but illustrates why intermediate calculations should be archived.

The highest observed reconnaissance discharge, 0.660 m³/s, was only 36.7% of the operational design runoff of 1.80 m³/s. This difference is expected because field observations were not collected during the design storm. It should not be interpreted as evidence that the existing channel is safe. A reliable capacity assessment requires a surveyed longitudinal profile, stage-discharge relation, bankfull geometry, structures, culverts, downstream water levels, and extreme-event observations.

3.2 Satellite rainfall performance

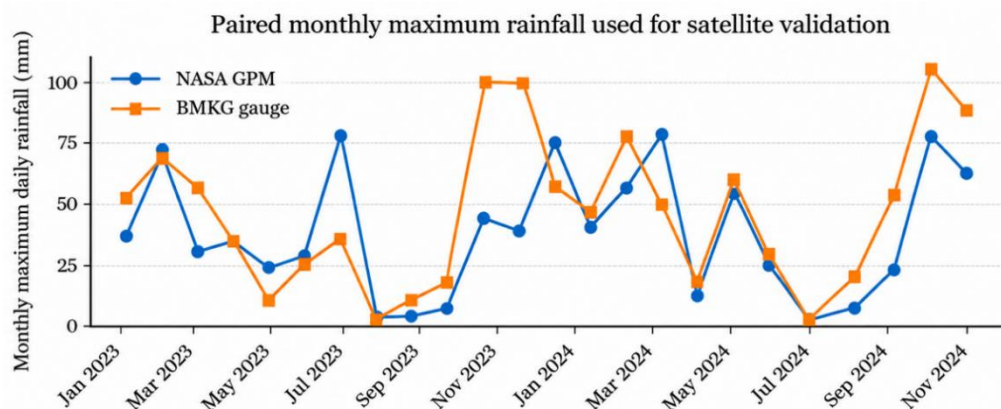


Fig. 2. Monthly maximum daily rainfall from NASA GPM and the nearby Meteorology, Climatology, and Geophysics Agency/Badan Meteorologi, Klimatologi, dan Geofisika (BMKG) gauge for the tabulated 2023–2024 overlap

NASA GPM reproduced part of the temporal variability, with $r = 0.68$, but it underestimated the gauge record in aggregate. The mean bias was -8.98 mm and PBIAS was -19.82% . The MAE of 18.13 mm and RMSE of 24.75 mm indicate substantial event-scale uncertainty for monthly maxima. The difference between RMSE and MAE shows that several large errors influenced performance, especially in November and December 2023, when gauge maxima approached 100 mm but satellite estimates remained below 43 mm. Conversely, NASA exceeded the gauge value markedly in July 2023.

The NSE of 0.36 means that the satellite series performed better than using the gauge mean but retained considerable unexplained variance. This is consistent with the wider literature: IMERG can be useful for spatial and daily-scale analysis in Indonesia, yet performance depends on season, terrain, and rainfall intensity (Ramadhan et al., 2022). For drainage design, bias correction or blended gauge–satellite methods are preferable where a reliable gauge overlap exists. More importantly, validation should target event duration and extremes relevant to the time of concentration rather than only monthly maxima.

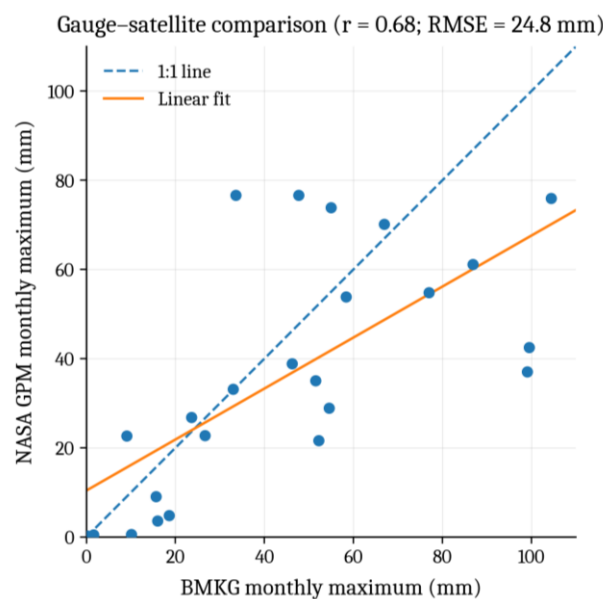


Fig. 3. Gauge–satellite agreement for paired monthly maxima. The dashed line is perfect agreement

The source report described the satellite–gauge agreement as good because MAE was below 20 mm and RMSE was not much larger. That interpretation is too categorical. No universal threshold makes an 18 mm MAE acceptable for all design purposes. Relative to a 50 mm heavy-rain threshold, the MAE equals 36% . Relative to the 10-year intensity calculation, underestimation of short-duration extremes could materially alter design discharge. The data can support preliminary analysis, but their uncertainty should be propagated rather than treated as negligible.

A simple sensitivity calculation illustrates the design consequence. The aggregate satellite total was 19.82% below the gauge total. Applying the reciprocal of that aggregate ratio as an illustrative adjustment would raise the reported 7.34 mm/h intensity to approximately 9.15 mm/h. With the same 2.52 km² catchment and operational runoff coefficient of 0.35 , the rational method would yield about 2.24 m³/s rather than 1.80 m³/s. This value exceeds the stated channel capacity of 1.90 m³/s by approximately 0.34 m³/s. This calculation is an illustrative stress test rather than a formal bias-correction model or probabilistic uncertainty analysis: monthly maxima and short-duration intensities are not equivalent, the overlap sample is small, and no calibrated event-based model is available. The result therefore demonstrates sensitivity of the pass-fail conclusion to rainfall uncertainty, but it should not be interpreted as a prediction of the corrected design

discharge or the probability of channel failure. A defensible correction should be developed by duration and season using a longer overlap, quantile mapping or event-based scaling, and independent validation.

Table 2. Validation statistics for NASA GPM monthly maximum rainfall

Metric	Value	Interpretation
Mean bias	-8.98 mm	Average NASA minus gauge difference
Mean absolute error	18.13 mm	Average absolute event-scale discrepancy
Root mean square error	24.75 mm	Error weighted toward large deviations
Pearson correlation	0.68	Moderate positive co-variation
Percent bias	-19.82%	Aggregate satellite underestimation
Nash-Sutcliffe efficiency	0.36	Limited predictive efficiency for monthly maxima

3.3 Extreme-rainfall analysis

The partial-duration screening selected 30 daily maxima above 50 mm. The largest value was 234.21 mm, followed by 171.18 and 169.34 mm. Most retained events clustered between 50 and 85 mm, creating a strongly right-skewed sample. Peaks-over-threshold analysis can use more information than annual maxima, but it requires a defensible threshold and event independence. The project document did not report declustering criteria, storm separation time, stationarity testing, or goodness-of-fit diagnostics. These omissions limit statistical interpretation.

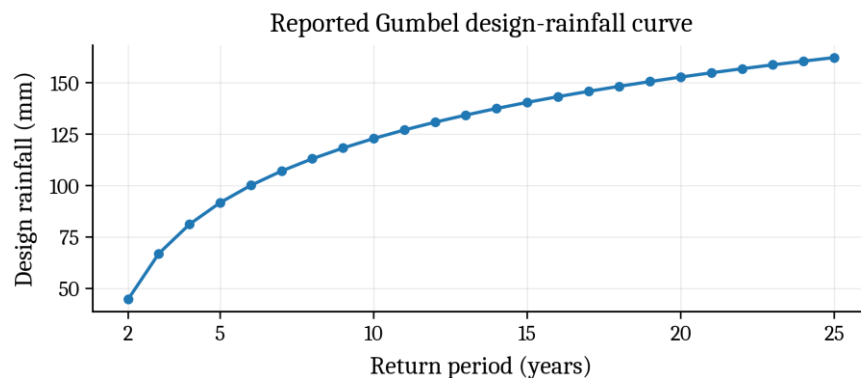


Fig. 4. Reported relationship between return period and Gumbel design rainfall

The reported design rainfall rose from 44.69 mm for 2 years to 162.28 mm for 25 years. Table 3 now includes the non-exceedance probability F and reduced variate $y(T)$ used to reconstruct the reported Gumbel curve. The equivalent Gumbel parameters are $\mu = 29.47$ mm and $\beta = 41.52$ mm. For the 10-year event, $y(10) = 2.2504$, yielding $X(10) = 29.47 + 41.52(2.2504) = 122.91$ mm. The corresponding reported intensity of 7.34 mm/h implies $t_c = 13.99$ h through the Mononobe relationship. However, because the original Kirpich flow-path length and catchment slope/elevation difference are absent from the source material, this duration cannot be confirmed as the result of the stated Kirpich calculation. This distinction is important: the audit can reproduce the Gumbel design-rainfall curve and the effective duration implied by Mononobe, but it cannot reconstruct undocumented Kirpich inputs.

Table 3. Reconstructed Gumbel intermediate parameters and reported rainfall-intensity values

Return period (years)	$F = 1 - 1/T$	Reduced variate $y(T)$	Design rainfall (mm)	Intensity (mm/h)
2	0.5000	0.3665	44.69	2.67
5	0.8000	1.4999	91.75	5.48
10	0.9000	2.2504	122.91	7.34
15	0.9333	2.6738	140.49	8.39
20	0.9500	2.9702	152.80	9.12
25	0.9600	3.1985	162.28	9.69

The source narrative states that design rainfall was calculated from 15 years of data, but the supplied dated rainfall record contains only 10 years, 2015–2024. For this reproducibility audit, 2015–2024 is therefore fixed as the operative extreme-rainfall record; the unsupported 15-year statement is excluded from the reconstructed analysis unless the missing five years of quality-controlled data are supplied. This distinction does not make the short record adequate for long return periods: a 10-year series is especially weak for estimating 20- or 25-year events, and sampling uncertainty increases rapidly as the target return period approaches or exceeds the record length. Gumbel analysis remains common, but it should not be treated as an a priori choice. For this short, strongly right-skewed record, model selection should compare Gumbel with generalized extreme value and Log-Pearson type III and, if a properly declustered partial-duration series can be established, a generalized Pareto peaks-over-threshold model; regional L-moment estimates may also help assess sensitivity to short at-site records (Coles, 2001; England et al., 2018; Hosking & Wallis, 1997; Rahayu et al., 2025). Goodness-of-fit diagnostics should consider both overall fit and upper-tail performance because the design decision depends on rare-event quantiles rather than central values. The final design should use the longest quality-controlled gauge series available, document event independence and threshold selection where applicable, and test nonstationarity where land use or climate conditions are changing.

3.4 Runoff increase from construction to operation

Using the reported inputs, construction runoff was 1.29 m³/s and operational runoff was 1.80 m³/s. The operational increase was 0.51 m³/s or 39.5%, driven entirely by the runoff-coefficient increase from 0.25 to 0.35. This result is directionally reasonable because housing development increases connected impervious area. However, a single composite coefficient can conceal substantial spatial variation. Roads and roofs may have high effective coefficients, while landscaped areas may retain infiltration if compaction is controlled. The coefficient should therefore be area-weighted from an explicit post-development land-cover plan.

Table 4. Rational method runoff calculations

Development stage	Area (km ²)	Intensity (mm/h)	Runoff coefficient	Peak Q (m ³ /s)
Construction	2.52	7.34	0.25	1.29
Operation	2.52	7.34	0.35	1.80

The hydraulic conclusion is sensitive to modest coefficient changes. With all other inputs unchanged, $C = 0.30$ gives approximately 1.54 m³/s, while $C = 0.40$ gives approximately 2.06 m³/s. The runoff coefficient at which the 1.90 m³/s channel capacity is exactly reached is about 0.369. Therefore, an increase of only 0.019 above the assumed operational coefficient, equivalent to about 5.4% relative change, removes the nominal capacity margin. These coefficient perturbations are illustrative deterministic scenarios rather than calibrated or validated predictions of post-development runoff. Their purpose is to identify how close the nominal design is to a decision threshold, not to predict the actual future runoff coefficient or discharge. Such a change could arise from a larger

connected roof and pavement fraction, compaction of landscaped soil, blocked infiltration routes, or future extensions. The design report should document the area and coefficient assigned to every land-cover class and distinguish total impervious area from hydraulically connected impervious area. It should also test wet antecedent conditions and post-construction soil properties. Where the final site plan is not fixed, the channel should be checked against a conservative upper-bound coefficient rather than the current point estimate.

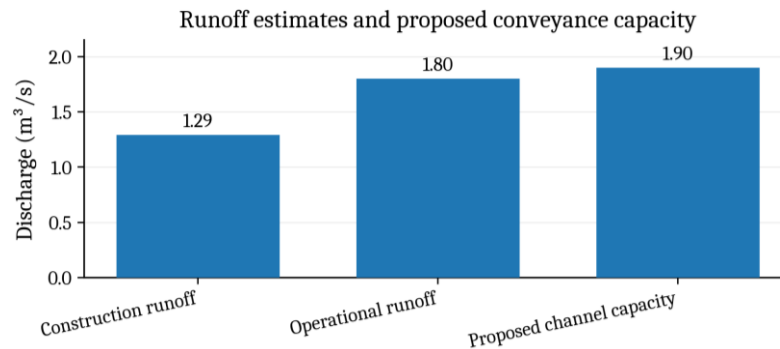


Fig. 5. Calculated construction and operational runoff compared with stated channel capacity

The 2.52 km² contributing area is far larger than the reported 0.29 km² project parcel used for the sedimentation-pond calculation. This distinction is critical. A conveyance channel that receives external catchment runoff must be designed for the full upstream area, while on-site detention may only intercept project-generated runoff. The manuscript therefore retains both areas for their respective calculations but recommends a clearly diagrammed catchment-routing scheme showing which flows enter the pond, which bypass it, and how they combine before discharge to the Tamangapa Canal.

The project report separately states a peak creek discharge of 22.377 m³/s based on maximum rainfall and contributing area, but it does not provide the governing rainfall intensity or duration, return period, runoff coefficient or transformation method, exact contributing area, routing assumptions, or intermediate calculations. Accordingly, the 22.377 m³/s value cannot be independently reproduced and is not numerically reconciled, averaged, or substituted for the reproducible 1.80 m³/s rational method result in this audit. It is classified as an unresolved source-project value and excluded from the reproducible primary design chain until its original calculation sheet and catchment routing are supplied. This creates a clear engineering disposition rather than an invented correction: if the missing inputs confirm 22.377 m³/s as the governing upstream design inflow, the stated 1.90 m³/s channel capacity is inadequate and the conveyance system requires redesign; if the value cannot be substantiated, the source project report should formally correct or withdraw it before final approval.

3.5 Open-channel capacity and reliability

The proposed capacity exceeds the operational design discharge by 0.10 m³/s, equivalent to 5.6%. Numerically, the design passes the stated deterministic comparison, but the 5.6% difference is not treated in this audit as an acceptable engineering safety factor or final-approval margin. It is smaller than the documented sensitivity to rainfall, runoff coefficient, roughness, blockage, sediment, and tailwater assumptions, so a modest reduction in effective area or increase in roughness can remove the surplus. Sediment deposition, weed growth, debris blockage, inlet contraction, bends, local structures, downstream backwater, and construction tolerance are not represented in the simple comparison. The 0.13 m freeboard is also limited for an open channel receiving runoff from a large mixed catchment. The channel is therefore classified as conditionally adequate only

under the nominal inputs and insufficiently robust for final engineering approval until positive capacity and freeboard remain after the validated uncertainty and operational allowances are applied.

Table 5. Reported trapezoidal-channel design parameters

Parameter	Reported value
Design discharge	1.80 m ³ /s
Bottom width	1.03 m
Top width	2.22 m
Design water depth	0.89 m
Total channel depth	1.03 m
Freeboard	0.13 m
Hydraulic radius	0.45 m
Bed slope	0.5%
Mean velocity	1.38 m/s
Stated capacity	1.90 m ³ /s
Flow area at design depth	1.446 m ² (reconstructed)
Manning roughness n	0.030 (back-calculated from R, S, and v)
Equivalent n from stated capacity	0.032 (using Q = 1.90 m ³ /s)

An illustrative derating further clarifies the risk. If blockage, vegetation, or sediment reduced effective conveyance by 10%, usable capacity would fall to approximately 1.71 m³/s, already below the 1.80 m³/s design flow; a 20% reduction would lower it to about 1.52 m³/s. These percentages are scenario-based stress tests, not calibrated estimates of capacity-loss probability. Their role is to demonstrate that the existing 5.6% nominal surplus is smaller than plausible operational deterioration and therefore cannot serve as the approval reserve. Final design should either increase physical conveyance capacity or reduce the verified design inflow until the validated uncertainty envelope, maintenance condition, blockage allowance, sediment accumulation, and tailwater effects still leave positive capacity and freeboard. The design should identify permissible sediment depth and vegetation condition, inspection intervals, cleaning triggers, and the responsible maintenance entity. Access for excavation must be preserved along the channel. Critical transitions, culverts, bends, and the canal outfall require separate local-loss and backwater checks. The channel should also be examined for erosive velocity and lining stability.

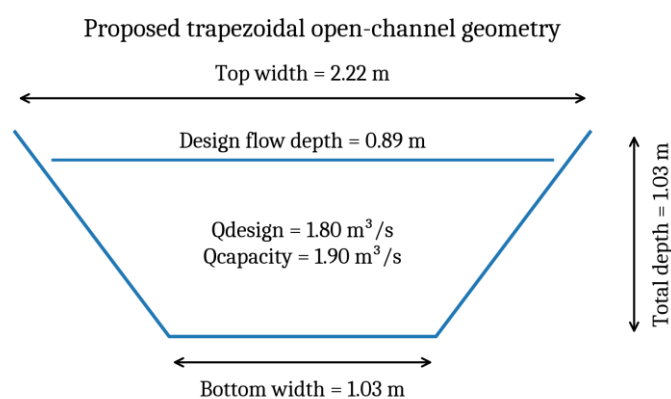


Fig. 6. Simplified geometry reconstructed from the reported channel dimensions

The source table reports a bed slope of 0.5%, while the original design illustration appears to display a different slope notation. Although the source does not state Manning roughness, the reported hydraulic radius (0.45 m), slope (0.005), and mean velocity (1.38 m/s) imply $n = 0.030$. Using the stated capacity of 1.90 m³/s with the reconstructed flow area of 1.446 m² instead implies $n = 0.032$. This range (approximately 0.030–0.032) is narrow but demonstrates that the published velocity and capacity are not exactly derived

from one fully documented Manning calculation. The reconstructed value $n \approx 0.030$ is therefore reported for transparency, while final engineering verification still requires confirmation of the original roughness selection, signed hydraulic calculation sheet, longitudinal profile, typical sections, material specification, and downstream boundary condition. Current drainage guidance emphasizes performance across the complete system rather than only the nominal cross-section (Federal Highway Administration, 2024).

A safer design approach would evaluate several scenarios; baseline roughness, vegetation or sediment-increased roughness, partial blockage, higher runoff coefficients, rainfall bias correction, and a larger return period. Dynamic simulation with storm water management model (SWMM) or an equivalent model could represent storage, routing, inlet controls, and external catchments (Rossman & Huber, 2016; Rossman & Simon, 2022). Green infrastructure should complement rather than replace conveyance. Distributed bioretention, permeable pavement, rainwater harvesting, and controlled roof discharge can reduce the operational peak and restore part of the pre-development water balance (Eckart et al., 2017; Palermo et al., 2019).

3.6 Sedimentation-pond storage

The source volume of $12,960 \text{ m}^3$ is correct only when eight hours is converted to 28,800 seconds. The written value of 43,200 seconds corresponds to 12 hours and would require $19,440 \text{ m}^3$ at $0.45 \text{ m}^3/\text{s}$. The recommended rectangular volume of $13,000 \text{ m}^3$ exceeds the corrected eight-hour hydraulic requirement by only 40 m^3 , or 0.31%. This 0.31% is a nominal geometric difference, not a usable engineering reserve, because it does not include side slopes, sediment accumulation, dead storage, outlet submergence, freeboard, survey tolerance, or inflow uncertainty. The pond is therefore classified as insufficient for final approval in its currently documented form. The final design must demonstrate active detention storage separately from sediment/dead storage and freeboard and must retain positive storage across the validated inflow and geometric uncertainty envelope.

Table 6. Corrected sedimentation-pond storage check

Parameter	Value
Design inflow	$0.45 \text{ m}^3/\text{s}$
Correct detention time	8 h = 28,800 s
Required hydraulic volume	$12,960 \text{ m}^3$
Plan dimensions	100 m $\times$ 52 m
Nominal depth	2.5 m
Provided rectangular volume	$13,000 \text{ m}^3$
Nominal geometric surplus	40 m^3 or 0.31%; not an approval reserve

Even a modest allowance changes the required dimensions. A 10% operational and sediment reserve above the eight-hour hydraulic volume would require $14,256 \text{ m}^3$, while a 20% reserve would require $15,552 \text{ m}^3$. Both exceed the proposed $13,000 \text{ m}^3$. If the 43,200-second value reflected an intended 12-hour detention criterion rather than a typographical error, the hydraulic requirement would be $19,440 \text{ m}^3$ before reserves. The design must therefore state whether eight or 12 hours is the governing target and separate active detention storage from permanent sediment storage and freeboard. The effective volume should be computed from surveyed side slopes and operating water levels, not simply length multiplied by width and total excavation depth. Outlet hydraulics should confirm that the target detention time occurs across the design range, because a nominal geometric volume does not guarantee hydraulic residence time when short-circuiting or an oversized outlet is present.

A sedimentation facility must have separate hydraulic and sediment-storage functions. The permanent or temporary sediment zone reduces over time as material accumulates. Maintenance access, a forebay, energy dissipation, emergency spillway, stabilized outlet, and safe drawdown are also necessary. Stormwater-pond sediments can contain metals and

other particulate and organic contaminants, so removal and disposal should follow characterization and environmental procedures (Flanagan et al., 2021). Detention facilities designed for combined flood and sediment control require explicit outlet and sediment-trap criteria (Suripin et al., 2020). First-flush dynamics may deliver a disproportionate mass early in an event, which supports using a forebay and treatment train rather than a single undivided basin (Maniquiz-Redillas et al., 2022).

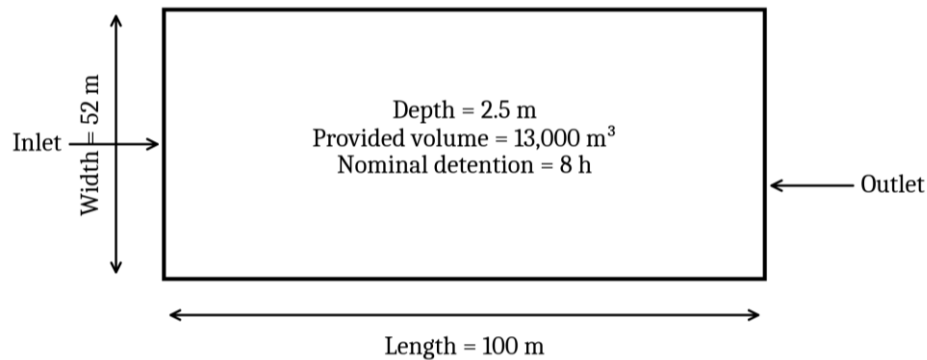


Fig. 7. Plan-view reconstruction of the recommended sedimentation pond

The pond calculation uses a runoff coefficient of 0.7 and a 0.29 km² project catchment to obtain an inflow of 0.45 m³/s, but the corresponding rainfall intensity is not shown. Reconstructing the rational method implies an intensity of approximately 8.0 mm/h, which differs from the 7.34 mm/h used for the full-catchment channel. The difference may be due to rounding or a different design event. The final design should use one documented rainfall basis or explicitly justify separate criteria.

3.7 Sediment load and receiving-water implications

At a constant assumed flow of 1.80 m³/s, TSS increased from 8.0 mg/L upstream to 9.0 mg/L downstream. The corresponding sediment load increased from 1.25 to 1.40 t/day, a rise of approximately 12%. The arithmetic is internally consistent with $L = 0.0864QC$. However, the values represent instantaneous concentrations combined with a design discharge, not a measured daily sediment hydrograph. Sediment transport is highly event-dependent, and concentrations typically change across the rising and falling limbs of a storm. The reported loads should therefore be interpreted as scenario estimates rather than annual or typical daily loads.

Table 7. Estimated suspended-sediment load along the Tamangapa Canal

Location	Q (m ³ /s)	TSS (mg/L)	Load (t/day)
Upstream	1.80	8.0	1.25
Middle	1.80	8.5	1.32
Downstream	1.80	9.0	1.40

Construction-phase erosion control should prioritize source reduction before pond treatment. Practical measures include limiting the area of simultaneous disturbance, rapidly stabilizing exposed soil, protecting stockpiles, using check dams or silt barriers, maintaining vegetated buffers, stabilizing entrances, and inspecting controls after rain. The pond should receive only flows for which it is designed. Clean external water should be diverted where feasible to avoid consuming treatment volume, while potentially sediment-laden project runoff should pass through a defined control train.

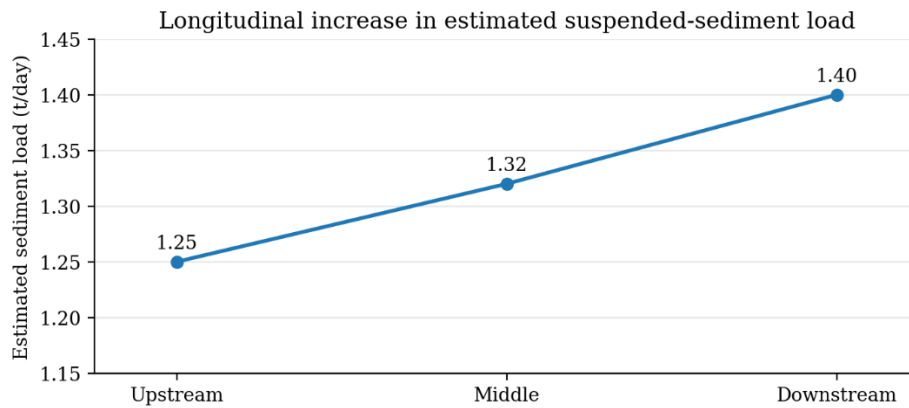


Fig. 8. Increase in calculated suspended-sediment load from upstream to downstream

3.8 Reproducibility audit and integrated design implications

The principal result of this manuscript is the reproducibility audit rather than the reconnaissance field observations themselves. Several individual calculations can be recomputed, but the complete design chain is only partially reproducible because key intermediate inputs, data-provenance details, and cross-checks are absent or internally inconsistent. Under the narrow set of reported assumptions, the project concept is hydraulically plausible; however, the interaction between the short rainfall record, satellite underestimation, incomplete frequency-analysis documentation, uncertain catchment routing, and minimal infrastructure margins can materially change system performance and, in some illustrative sensitivity cases, reverse the nominal pass-fail conclusion. These cases are diagnostic stress tests of assumption sensitivity, not fully validated predictions of future hydraulic performance.

The engineering uncertainties identified by the audit have direct implications for downstream communities. The drainage system discharges toward the Tamangapa Canal, and the analysis distinguishes a 2.52 km² contributing catchment from the much smaller project parcel. If peak runoff is underestimated, if external inflows are incompletely routed, or if channel and pond capacity deteriorates through sediment, vegetation, or blockage, excess water may be transferred to downstream reaches rather than safely retained within the development. The relevant consequence is not limited to whether the project channel overtops; it also includes the possibility of increased flood depth or duration, erosion and sediment accumulation, impaired local drainage, and additional maintenance demands beyond the project boundary. These pathways should therefore be evaluated as part of the connected creek-channel-pond-canal system rather than as an isolated site-drainage problem.

This broader framing introduces an environmental-justice consideration. A development may create benefits within its own boundary while transferring part of its residual stormwater risk to people who do not share those benefits. The present audit cannot determine whether such impacts are socially unequal because the source records do not include household locations, socioeconomic status, tenure, mobility constraints, critical-facility exposure, or other indicators of social vulnerability. Accordingly, environmental justice is treated here as a planning and governance implication rather than an empirical finding. Before final approval, downstream receptors and critical infrastructure should be mapped, cumulative flood and sediment pathways should be assessed, and residual risk, maintenance responsibility, and emergency response arrangements should be made transparent. Where downstream residents or public facilities could be affected, their concerns should be incorporated into monitoring and adaptive-management decisions so that project-boundary accounting does not externalize flood risk.

The remaining data inconsistencies are resolved in this audit by explicit source-handling rules rather than by inventing missing values. The 10-year 2015–2024 rainfall

series is the operative record for the reconstructed frequency analysis, and the dated 2023–2024 monthly pairs are the operative gauge–satellite validation overlap. The unsupported 15-year and 2024–2025 narrative labels are not used unless additional raw records are supplied. The 22.377 m³/s peak discharge is treated as an unresolved source-project value and excluded from the reproducible primary design chain until its original inputs are disclosed; confirmation of that value would require redesign because it far exceeds the stated 1.90 m³/s channel capacity, whereas failure to substantiate it requires correction of the source report. Likewise, the 5.6% channel surplus and 0.31% pond surplus are not accepted as safety factors: both are classified as insufficient for final engineering approval under the uncertainty documented in this audit.

Table 8. Principal reproducibility, reconciliation, and design-control issues

Issue	Observed inconsistency or gap	Required action
Data period	Narrative states 15 years; supplied dated series contains 10 years (2015–2024)	Use 2015–2024 as the operative audit record; exclude the 15-year claim unless the missing dated series is supplied
Gauge overlap	Narrative states 2024–2025; 24 numerical pairs are dated 2023–2024	Use 2023–2024 for validation; do not use the 2024–2025 label unless raw dated records substantiate it
Peak discharge	22.377 m ³ /s lacks reproducible inputs and conflicts with the documented 1.80 m ³ /s design chain	Exclude 22.377 m ³ /s from the reproducible primary design chain pending source calculation; if confirmed as governing, redesign the 1.90 m ³ /s channel; otherwise correct the source report
Time conversion	8 h written as 43,200 s	Use 28,800 s; 43,200 s equals 12 h
Channel hydraulics	Original Manning n absent; audit back-calculates $n \approx 0.030$ – 0.032 from reported values	Confirm the original design n, profile, controls, and tailwater
Design margin	Channel capacity is only 5.6% above nominal design flow and the surplus disappears under modest stress scenarios	Treat 5.6% as insufficient for final approval; increase capacity or reduce verified inflow until positive capacity/freeboard remains across validated uncertainty and operational allowances
Pond reserve	Nominal geometric surplus is only 0.31% and excludes sediment/dead storage, freeboard, and uncertainty	Treat 0.31% as insufficient; resize or reconfigure the pond so active detention plus sediment/dead storage and freeboard remain adequate across the validated uncertainty envelope
Frequency and timing inputs	Original Kirpich L and slope/elevation inputs and Gumbel intermediate statistics are absent	Audit reconstructs $\mu = 29.47$ mm, $\beta = 41.52$ mm and implied $t_c = 13.99$ h; original Kirpich inputs require source confirmation
Approval accuracy	No universal single percentage threshold; current margins are smaller than documented uncertainty	Withhold final approval while any unresolved value can change the pass–fail decision; require traceability, independent model validation, and positive capacity/storage/freeboard across the validated uncertainty envelope

Acceptable design accuracy before engineering approval should therefore be defined at the decision level rather than by arithmetic agreement alone. All governing hydrological and hydraulic calculations should be fully traceable and independently reproducible, with no unresolved discrepancy capable of changing design discharge, water level, storage, or the pass–fail conclusion. If a rainfall–runoff model is used for final verification, calibration and independent validation should at minimum reach commonly used satisfactory flow-performance screening benchmarks— $R^2 > 0.60$, $NSE > 0.50$, and $|PBIAS| \leq 15\%$ —while

recognizing that these are not Indonesian regulatory acceptance limits (Moriassi et al., 2015). Most importantly, the validated uncertainty range in inflow, stage, and effective capacity must remain smaller than the available engineering reserve. The channel, pond, and downstream system should retain positive capacity, storage, and freeboard under the governing design event after uncertainty, tailwater, blockage, sediment, roughness, maintenance condition, and construction tolerances are considered. If the uncertainty envelope crosses the failure threshold, final approval should be withheld until uncertainty is reduced or the physical design margin is increased.

A staged verification process is recommended. First, establish a single geospatial catchment model with subarea boundaries, land cover, elevations, and all inlet and outlet paths. Second, extend and quality-control gauge rainfall, correct satellite bias, and construct intensity–duration–frequency curves for relevant durations. Third, calibrate or at least benchmark runoff using observed water levels and flow during several storms. Fourth, model the channel, pond, culverts, and Tamangapa Canal boundary as one connected system. Fifth, incorporate source controls and assess performance under maintenance and blockage scenarios. Finally, establish construction and operational monitoring thresholds for rainfall, water level, overtopping, sediment accumulation, and inspection frequency.

The monitoring plan should translate these steps into measurable controls. A recording rain gauge should be installed near the site and checked against the BMKG station. Water-level loggers at the upstream creek, channel outlet, and pond can document lag time, peak stage, and drawdown. Staff gauges and fixed cross-section markers should support rapid post-storm inspection. Sediment-depth plates in the forebay and main pond should define a removal threshold before storage is materially lost. Inspections should occur before the wet season, after major rainfall, and whenever water remains beyond the target drawdown period. Construction records should document disturbed area, stabilization status, and maintenance actions. These data would create an auditable link between rainfall, runoff, facility performance, and corrective action, allowing the preliminary model to be updated rather than remaining a one-time calculation. Monitoring results should also be communicated in a form accessible to local authorities and potentially affected downstream communities, including records of overtopping, prolonged ponding, sediment removal, maintenance failures, and corrective actions. This creates a procedural-equity mechanism alongside the technical monitoring program.

Adaptive management should be linked to predefined decision thresholds. For example, repeated water levels close to freeboard, drawdown longer than the design period, or sediment occupying a specified fraction of storage should trigger engineering review rather than routine cleaning alone. Rainfall and stage records can also be used to estimate event runoff coefficients and test whether the assumed value of 0.35 remains representative after development. If monitoring shows a higher coefficient or faster concentration time, distributed controls or additional storage should be installed before recurrent overtopping occurs. This feedback mechanism converts monitoring from a compliance formality into a design-verification process and provides evidence for future revisions of the drainage model.

The approach is consistent with integrated sustainable drainage. Distributed controls can reduce inflow to the main channel, while the channel and pond provide residual conveyance and storage. No single measure should carry the entire reliability burden. International evidence shows that low-impact development can reduce runoff volume and peaks, but performance remains site-specific and must be supported by design, monitoring, and maintenance (Dietz, 2007; Ahiablame et al., 2012; Hamel et al., 2013; Liu et al., 2015; Fletcher et al., 2015; Eckart et al., 2017; Golden & Hoghooghi, 2018).

3.9 Study limitations

Because this manuscript is a secondary analysis and reproducibility audit, its findings are constrained by the quantity, resolution, and provenance of the source material. The audit has resolved the rainfall-period labeling for analytical purposes by using only the

supplied dated records—2015–2024 for the extreme-rainfall series and 2023–2024 for the gauge–satellite overlap—but this does not recover the missing five years implied by the unsupported 15-year statement. The 10-year rainfall record remains short relative to the 20- and 25-year return periods, so sampling uncertainty in the upper tail is substantial and the fitted extreme-rainfall curve should not be treated as a stable long-term estimate. Although equivalent Gumbel parameters can be reconstructed from the published return-period curve, the original sample statistics and complete frequency-analysis worksheet are not available. Second, the 22.377 m³/s peak-flow value remains source-unresolved because its calculation inputs are absent; it is excluded from the reproducible design chain, but final engineering approval still requires either source verification and consequent redesign or formal correction of that value.

Third, creek discharge is based on only a small number of reconnaissance-level measurements from two survey periods. These observations were not collected during design storms and do not establish a stage-discharge rating curve, seasonal flow regime, or flood-frequency relationship; therefore, they provide contextual field evidence rather than independent validation of design discharge or channel safety. Fourth, the sensitivity analyses in this manuscript are deliberately illustrative. The rainfall-bias adjustment, runoff-coefficient perturbations, and 10–20% conveyance derating are deterministic stress tests selected to show whether modest changes in uncertain assumptions can alter the nominal pass-fail conclusion. They are not calibrated hydrological or hydraulic simulations, formal bias-correction procedures, Monte Carlo uncertainty analyses, or estimates of failure probability, and the specific scenario magnitudes should not be interpreted as predicted future conditions. Additional limitations include the absence of the original Kirpich flow-path length and catchment slope/elevation difference, the lack of a documented Manning roughness coefficient, unreported downstream boundary conditions and structural losses, simplified rectangular pond geometry, and the absence of particle-size and hydraulic-efficiency data for sediment-removal assessment. Consequently, the results are appropriate for reproducibility assessment, preliminary design review, and identification of decision-sensitive assumptions, but not for final hydraulic certification. A further limitation is that the source material contains no household-level flood exposure, demographic, income, tenure, health, mobility, or social-vulnerability information for communities along the downstream creek and Tamangapa Canal. The manuscript therefore cannot quantify distributive or procedural inequity, identify disproportionately affected groups, or attribute observed community impacts to the development. The environmental-justice discussion should be interpreted as a risk-governance implication that identifies data and engagement needs for subsequent assessment, not as evidence of an existing inequitable impact.

4. Conclusions

This reproducibility audit indicates that the project hydrology package is appropriate only as a preliminary design basis, not for final hydraulic certification. For analytical consistency, the audit fixes the available rainfall record at 2015–2024 and the gauge–satellite overlap at 2023–2024, because these are the dated records supplied. The unexplained 22.377 m³/s peak discharge is excluded from the reproducible design chain pending source verification; if confirmed as governing, the current channel requires redesign. The 5.6% channel margin and 0.31% pond surplus are likewise treated as insufficient for final approval because documented uncertainty can eliminate both margins. Final engineering approval therefore requires reconciled governing inputs, independently reproducible and validated hydrological–hydraulic calculations, and demonstrated positive channel capacity, pond storage, and freeboard across the validated uncertainty envelope for the connected creek–channel–pond–canal system.

Acknowledgement

The authors acknowledge Limited Liability Company/*Perseroan Terbatas* (PT) Enviro Celebes Konsultan for providing access to the project database and supporting information

used in this reproducibility audit. PT Bumi Surya Persada is cited as the source organization of the unpublished project hydrology report examined in this study. Neither organization is listed as an author or research funder of this manuscript. The analytical procedures, interpretation of results, and manuscript preparation were undertaken by the listed authors, as specified in the author contribution statement; the consultant's documented role in this study was limited to facilitating access to the database and supporting information. Appreciation is extended to the Government of Gowa Regency for its support and facilitation during the research process, and to the Institute for Research and Community Service/*Lembaga Penelitian dan Pengabdian kepada Masyarakat* (LPPM) Universitas Halu Oleo, Institut Kesehatan dan Teknologi Buton Raya, and Institut Teknologi dan Kesehatan Tri Tunas Nasional for institutional, coordination, and administrative support.

Author Contribution

Conceptualization, M.S. and C.D.; Methodology, M.S.; Software, A.M.D.; Validation, C.D.; Formal Analysis, C.D.; Investigation, A.M.D.; Resources, M.S.; Data Curation, C.D.; Writing – Original Draft Preparation, M.J.S.; Writing – Review & Editing, C.D.; Visualization, A.M.D.; Supervision, M.S.; Project Administration, M.S.

Funding

This research received no external funding.

Ethical Review Board Statement

Not available. The study used environmental monitoring data and did not involve human participants or animals.

Informed Consent Statement

Not available.

Data Availability Statement

The observations and design inputs analyzed in this reproducibility audit are contained in the existing project hydrology report and associated records. No new field dataset was generated specifically for this manuscript. Access to the project database and supporting information was facilitated by PT Enviro Celebes Konsultan. Additional raw data and calculation sheets may be obtained from the corresponding author subject to permission from the data owner.

Conflicts of Interest

The authors declare no conflict of interest. PT Enviro Celebes Konsultan provided access to the project database and supporting information

Declaration of Generative AI Use

During the preparation of this work, the authors used OpenAI ChatGPT to assist with English-language editing, manuscript restructuring, calculation checking, and formatting to the journal template. The authors reviewed and edited all content, verified the numerical results against the source data, and take full responsibility for the publication.

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