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Hydraulic Simulation of Flood Water Levels of the Saddang River from Upstream of Tagari Bridge to Singki' Bridge in North Toraja Regency Using HEC-RAS Software

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Abstract

The Saddang River is one of the major rivers in North Toraja Regency, South Sulawesi, which in recent years has frequently overflowed, particularly in urban areas and along the riverbanks, causing damage to public facilities, plantations, rice fields, and residential areas, as well as bank erosion that threatens surrounding infrastructure. This study aims to calculate the design flood discharge of the Saddang River for return periods of 2, 5, 10, and 25 years and to simulate the flood water level profile along a river segment of approximately 2.5 km, from upstream of Tagari Bridge to Singki' Bridge. The design rainfall analysis was carried out using the Gumbel method based on annual maximum daily rainfall data from the Makale Station for the period 2015–2024, with goodness-of-fit verified through the Chi-Square test and the Smirnov-Kolmogorov test. The design flood discharge was calculated using the Nakayasu Synthetic Unit Hydrograph (HSS Nakayasu) method, while the water surface profile was simulated using HEC-RAS software under steady flow conditions. The results show that peak discharge increases with increasing return period, namely 1052.427 m³/s (Q2), 1343.124 m³/s (Q5), 1551.933 m³/s (Q10), and 1803.400 m³/s (Q25). The HEC-RAS simulation shows that this increase in design discharge is directly proportional to the rise in water level and the number of cross-sections (stations) experiencing overflow: the maximum water level increased from 7.37 m at Q2 to 8.22 m at Q5, 8.77 m at Q10, and 9.39 m at Q25, with stations 1, 2, 3, 5, 6, 7, 8, and 9 consistently experiencing overflow across all return periods tested. These findings indicate that the conveyance capacity of the Saddang River cross-section is no longer adequate in several segments to accommodate the design flood discharge, so that targeted flood management and control measures are needed in the segments prone to overflow.

Keywords: Saddang River, Flood, Water Level, Nakayasu Synthetic Unit Hydrograph, HEC-RAS

INTRODUCTION

Rivers play an important role in the ecosystem, serving as a source of raw water, a habitat for various living organisms, a means of transportation, and a source of hydroelectric power, so their existence must continually be maintained. As a conduit carrying water from upstream to downstream, a river channel requires adequate conveyance capacity to convey flow without causing overflow into the surrounding area.

Flooding occurs when the flow volume exceeds the storage or conveyance capacity of a river, lake, swamp, drainage system, or other waterway within a given period. Prolonged heavy rainfall can cause the volume of water that must be accommodated by a river channel or other waterway to exceed its capacity, causing the water to overflow and inundate the surrounding area. Communities living near rivers are the group most at risk from this threat; the closer a settlement is to a flood source, the greater the risk it faces. Rainfall intensity and duration are the primary factors causing floods, compounded by topography, soil conditions, and land cover within a given watershed (Daerah Aliran Sungai, DAS).

The Saddang River is one of the rivers that crosses North Toraja Regency. In recent years, overflows of the Saddang River have repeatedly occurred at several points, particularly in urban areas and along the riverbanks. Based on direct interviews with local residents, the heavy flows that recur annually due to intense rainfall have damaged public facilities, plantations, rice fields, and residential areas, a situation further aggravated by riverbank erosion that threatens important facilities nearby. This makes the segment of the Saddang River from upstream of Tagari Bridge to Singki' Bridge an area requiring special attention.

Based on the foregoing, this study was carried out to analyze the flood water level in this segment using the HEC-RAS (Hydrologic Engineering Center's – River Analysis System) hydraulic modeling program, for return periods of 2, 5, 10, and 25 years.

IDENTIFY, RESEARCH AND COLLECT IDEA

Problem Statement

1. What is the design flood discharge of the Saddang River for return periods of 2, 5, 10, and 25 years?
2. What are the results of the water level simulation of the Saddang River using HEC-RAS software?

Research Objectives

1. To calculate the design flood discharge of the Saddang River for return periods of 2, 5, 10, and 25 years.
2. To analyze the flood water level simulation of the Saddang River using HEC-RAS software.

Scope and Limitations

1. Topographic and rainfall data used are the 2015-2024 records from BBWS Pompengan Jeneberang.
2. Water level simulation for return periods of 2, 5, 10, and 25 years was performed using HEC-RAS 4.1 software under steady flow conditions.

WRITE DOWN YOUR STUDIES AND FINDINGS

RESEARCH METHODOLOGY

Study Location

This study was conducted in the Saddang River watershed (DAS), North Toraja Regency, with a watershed area of 339 km² at the study segment and a river width of 30–60 m. The study area is limited to the segment of the Saddang River extending from upstream at Tagari to downstream at Singki, with a flow length of approximately 2.5 km. The watershed boundary was determined using satellite imagery via the Google Earth application, aided by a polygon feature following the natural topographic boundary.

Data and Data Sources

Mixture Variations

The data used in this study are secondary data, consisting of watershed characteristic data, topographic data, and annual maximum daily rainfall data from the Makale Station for the period 2015–2024, obtained from BBWS Pompengan Jeneberang. Because the rainfall data came from a single station with no missing records, areal rainfall computation using the Thiessen Polygon method and data correction were not required.

Analysis Stages

The analysis was carried out in four main stages. First, the preparation and collection of watershed characteristic, topographic, and nearby rainfall data. Second, frequency distribution analysis of rainfall to select the appropriate distribution type based on statistical parameters (standard deviation S_d , skewness coefficient C_s , kurtosis coefficient C_k , and coefficient of variation C_v), followed by a goodness-of-fit test (Chi-Square and Smirnov-Kolmogorov) and calculation of the design rainfall. Third, calculation of the design flood discharge using the Nakayasu Synthetic Unit Hydrograph (HSS Nakayasu) method. Fourth, hydraulic modeling with HEC-RAS, comprising input of river geometry data (exported from AutoCAD), input of cross-section data, input of design discharge data, and Steady Flow analysis to obtain the water level profile for each return period.

Manning roughness is an important component of the hydraulic calculation because channel resistance affects conveyance and energy losses. The HEC-RAS outputs include water-surface elevation, energy grade elevation, and energy slope for every modeled station. Consequently, differences in water level along the reach are not controlled by discharge alone. Local changes in channel width, depth, bank geometry, and roughness contribute to the variation of the computed profile.

The hydraulic simulation represents an approximately 2.5 km reach extending from the upstream area of Tagari Bridge to Singki' Bridge. Fifteen cross sections are represented in the model, with STA 15 as the upstream section and STA 1 as the downstream section. Riverbed elevation decreases from 842.00 m at STA 15 to 831.35 m at STA 1. The geometric data, including longitudinal and cross-sectional information, were prepared from topographic data and entered into HEC-RAS 4.1. The design discharges were then modeled as steady-flow profiles for Q2, Q5, Q10, and Q25.

HEC-RAS Model Configuration

The peak of the modeled hydrograph occurs at approximately the seventh hour for all four return periods. The peak discharge values are 1052.427 m³/s for Q2, 1343.124 m³/s for Q5, 1551.933 m³/s for Q10, and 1803.400 m³/s for Q25. The increase from Q2 to Q25 is 750.973 m³/s, equivalent to approximately 71.3%. The progressive increase confirms that the longer return-period rainfall scenarios generate increasingly severe hydraulic loading on the river cross sections. After the peak, the hydrograph decreases as runoff contribution and watershed storage diminish.

The selected design rainfall was transformed into design flood discharge using the Nakayasu Synthetic Unit Hydrograph (HSS Nakayasu) method. The principal watershed parameters used were an area of 339 km², a time lag (tg) of 2.621 hours, $\alpha = 1.914$, peak time (Tp) of 3.670 hours, and recession time T0.3 of 5.017 hours. The Nakayasu method relates effective rainfall to the response characteristics of the watershed and produces a design hydrograph that can be used as the hydraulic loading for the river model.

Synthetic Unit Hydrograph and Design Flood Discharge

The Gumbel parameters obtained in the thesis were $b = 85.80$ and $1/a = 23.819$. The design rainfall was calculated using $X_t = b + (1/a)Y_t$, where Y_t represents the reduced variate for the selected return period. The resulting design rainfall values are 94.534 mm for Q2, 121.531 mm for Q5, 139.402 mm for Q10, and 161.990 mm for Q25. The selection was checked statistically using Chi-Square and Smirnov–Kolmogorov tests. The calculated Chi-Square value of 2 is lower than the critical value of 5.991 at $\alpha = 5\%$ and 2 degrees of freedom. Likewise, $D_{max} = 0.12$ is lower than $D_{critical} = 0.409$. Both tests therefore support the adopted Gumbel distribution.

The annual maximum daily rainfall series from Makale Station was evaluated using statistical dispersion parameters. The ten observations for 2015–2024 range from 71 mm to 145 mm, with a mean rainfall of 97.7 mm and a standard deviation of 22.58 mm. The calculated skewness coefficient ($C_s = 0.781$), kurtosis coefficient ($C_k = 4.796$), and coefficient of variation ($C_v = 0.231$) were compared with the criteria for Normal, Log Normal, Log Pearson Type III, and Gumbel distributions. The results indicate that the Normal and Log Normal distributions do not satisfy the adopted criteria, whereas Log Pearson Type III and Gumbel are acceptable. Gumbel was selected for the subsequent frequency analysis.

A. Bits and Pieces together

RESULTS AND DISCUSSION

Watershed Characteristics

Table 1. Saddang River watershed data

No.	River Name	Watershed Area	River Width	River Length
1	Saddang River	339 km ²	30–60 m	38.3 km

Administratively, the Saddang River is located in North Toraja Regency, South Sulawesi Province. The study segment in this research is limited to approximately 2.5 km, from upstream of Tagari Bridge to Singki' Bridge, with a watershed area at that segment of 339 km².

Rainfall Data and Design Rainfall Analysis

Table 2. Annual maximum daily rainfall, Makale Station (2015–2024)

3	Year	Rainfall, Makale Station (mm)
1	2015	73
2	2016	96
3	2017	110
4	2018	103
5	2019	145
6	2020	71
7	2021	75
8	2022	100
9	2023	91
10	2024	114

Based on these ten years of rainfall data, dispersion statistical parameters were calculated to determine the appropriate distribution type.

Table 3. Dispersion statistical parameters of the rainfall data

Parameter	Value
Mean rainfall ($\bar{X}$)	97.7 mm
Standard deviation (Sd)	22.58
Skewness coefficient (C_s)	0.781
Kurtosis coefficient (C_k)	4.796
Coefficient of variation (C_v)	0.231

These parameter values were compared against the requirements for each distribution type (Normal, Log Normal, Log Pearson Type III, and Gumbel).

Table 4. Conformity of statistical parameters with distribution requirements

Distribution Type	Requirement	Remarks
Normal	$C_s \approx 0$ and $C_k \approx 3$	Not satisfied
Log Normal	$C_s/C_v \approx 3$	Not satisfied
Log Pearson Type III	$C_s \neq 0$	Satisfied
Gumbel	$C_s \leq 1.1396$ and $C_k \leq 5.4002$	Satisfied

Although the data parameters statistically satisfy the requirements of both the Log Pearson Type III and Gumbel distributions, the Gumbel distribution was selected for this study because it is a

method commonly used for extreme rainfall frequency analysis in Indonesia and is consistent with practice in similar hydrological studies. The suitability of the Gumbel distribution was then verified through two goodness-of-fit tests, namely the Chi-Square test and the Smirnov-Kolmogorov test.

The Chi-Square test produced a calculated Chi-Square value of 2, which is smaller than the critical Chi-Square value of 5.991 at 2 degrees of freedom and a 95% confidence level ($\alpha = 5\%$), so the Gumbel distribution can be accepted. The Smirnov-Kolmogorov test produced a maximum deviation value (Dmax) of 0.12, considerably smaller than the critical value (Dcritical) of 0.409 at the same significance level, so the Gumbel distribution is likewise accepted under this test. Both goodness-of-fit tests consistently confirm that the Gumbel distribution is appropriate for modeling the distribution of the maximum rainfall data at the study site.

Table 5. Design rainfall using the Gumbel method

Return Period (Years)	Yt	Design Rainfall (mm)
2	0.3665	94.534
5	1.4999	121.531
10	2.2502	139.402
25	3.1985	161.990

Design rainfall shows a consistent increase with increasing return period, from 94.534 mm at a 2-year return period to 161.990 mm at a 25-year return period. These values were subsequently used as the effective rainfall input in the calculation of the design flood discharge.

Design Flood Discharge Using the Nakayasu Synthetic Unit Hydrograph Method

The design flood discharge was calculated using the Nakayasu Synthetic Unit Hydrograph (HSS Nakayasu) method, with the following watershed parameters: watershed area of 339 km², main river length of 2.5 km, time lag (tg) of 2.621 hours, parameter α of 1.914, hydrograph rise time (Tp) of 3.670 hours, and recession time T0.3 of 5.017 hours. Based on this unit hydrograph, the peak design flood discharge for each return period is summarized in Table 6.

Table 6. Summary of peak design flood discharge using the Nakayasu method

Return Period (Years)	Peak Design Flood Discharge (m ³ /s)
2	1052.427
5	1343.124
10	1551.933
25	1803.400

Peak discharge for all return periods consistently occurred at hour 7 from the start of effective rainfall, indicating a relatively fast watershed response to rainfall. Peak discharge increased from 1052.427 m³/s at a 2-year return period to 1803.400 m³/s at a 25-year return period, an increase of about 71% across the range of

return periods studied. After reaching its peak, the discharge entered a gradual recession phase as surface runoff decreased and watershed storage was progressively depleted.

Flood Water Level Simulation Using HEC-RAS

The design flood discharge values in Table 6 were subsequently used as input data for the water surface profile simulation in HEC-RAS software, together with river geometry data (longitudinal and cross sections) and Manning roughness coefficients as additional parameters. The simulation was carried out under steady flow conditions for 15 review points (stations) along the study segment, from station 15 (furthest upstream, near Tagari Bridge) to station 1 (furthest downstream, near Singki' Bridge).

Table 7. Water surface elevation and flood water level per station – 2-year return period

Station	Riverbed Elevation (m)	Water Surface Elevation Q2 (m)	Water Level Q2 (m)
15	842.00	847.14	5.14
14	842.31	846.83	4.52
13	840.54	845.53	4.99
12	839.97	844.42	4.45
11	838.95	842.97	4.02
10	837.27	841.54	4.27
9	835.65	840.88	5.23
8	835.10	840.29	5.19
7	833.89	839.79	5.90
6	832.92	839.58	6.66
5	834.33	839.38	5.05
4	832.61	839.08	6.47
3	832.27	838.89	6.62
2	831.68	838.79	7.11
1	831.35	838.72	7.37

Table 8. Water surface elevation and flood water level per station – 5-year return period

Station	Riverbed Elevation (m)	Water Surface Elevation Q5 (m)	Water Level Q5 (m)
15	842.00	847.74	5.74
14	842.31	847.43	5.12
13	840.54	846.15	5.61
12	839.97	845.04	5.07

Station	Riverbed Elevation (m)	Water Surface Elevation Q5 (m)	Water Level Q5 (m)
11	838.95	843.62	4.67
10	837.27	842.32	5.05
9	835.65	841.72	6.07
8	835.10	841.16	6.06
7	833.89	840.67	6.78
6	832.92	840.46	7.54
5	834.33	840.27	5.94
4	832.61	839.93	7.32
3	832.27	839.73	7.46
2	831.68	839.64	7.96
1	831.35	839.57	8.22

Table 9. Water surface elevation and flood water level per station – 10-year return period

Station	Riverbed Elevation (m)	Water Surface Elevation Q10 (m)	Water Level Q10 (m)
15	842.00	848.16	6.16
14	842.31	847.83	5.52
13	840.54	846.56	6.02
12	839.97	845.45	5.48
11	838.95	844.05	5.10
10	837.27	842.83	5.56
9	835.65	842.25	6.60
8	835.10	841.70	6.60
7	833.89	841.21	7.32
6	832.92	841.00	8.08
5	834.33	840.80	6.47
4	832.61	840.48	7.87
3	832.27	840.28	8.01
2	831.68	840.19	8.51
1	831.35	840.12	8.77

Table 10. Water surface elevation and flood water level per station – 25-year return period

Station	Riverbed Elevation (m)	Water Surface Elevation Q25 (m)	Water Level Q25 (m)
15	842.00	848.61	6.61
14	842.31	848.28	5.97
13	840.54	847.03	6.49
12	839.97	845.92	5.95
11	838.95	844.55	5.60
10	837.27	843.40	6.13
9	835.65	842.85	7.20
8	835.10	842.30	7.20
7	833.89	841.81	7.92
6	832.92	841.60	8.68
5	834.33	841.40	7.07
4	832.61	841.09	8.48
3	832.27	840.90	8.63
2	831.68	840.81	9.13
1	831.35	840.74	9.39

Summary of Overflow Points and Spatial Flood Pattern

Table 11. Summary of overflowing stations and maximum water level per return period

Return Period	Stations Experiencing Overflow	Maximum Water Level (m)
Q2 (2 years)	1, 2, 3, 5, 6, 7, 8, 9, 13	7.37
Q5 (5 years)	1, 2, 3, 4, 5, 6, 7, 8, 9, 11	8.22
Q10 (10 years)	1, 2, 3, 5, 6, 7, 8, 9, 11, 13, 15	8.77
Q25 (25 years)	1, 2, 3, 5, 6, 7, 8, 9, 10, 11, 13, 15	9.39

The summary in Table 11 shows a consistent spatial pattern: stations 1, 2, 3, 5, 6, 7, 8, and 9 experience overflow across all return periods tested, making them the most critical segment along the studied reach of the Saddang River. As the return period increases, the number of overflowing stations also increases — station 4 begins to be affected at Q5, while stations 10, 11, 13, and 15 begin or resume being affected at Q10 and Q25 — confirming that the conveyance capacity of the cross-section at this segment becomes increasingly inadequate as the design discharge rises. The maximum water level consistently occurs at station 1, the furthest

downstream point in the study segment, which increased from 7.37 m (Q2) to 8.22 m (Q5), 8.77 m (Q10), and 9.39 m (Q25).

Relationship Between Design Discharge and Water Level Rise

The simulation results confirm a direct relationship between design flood discharge and water level: a 71% increase in discharge from Q2 to Q25 was accompanied by an approximately 27% rise in maximum water level, from 7.37 m to 9.39 m. This difference in the rate of increase is consistent with the geometry of a natural river cross-section, which generally widens toward the banks, so that the rise in water level does not increase strictly linearly with discharge, even though the direction of the relationship remains clear and consistent. The energy grade line slope (E.G. Slope) from the HEC-RAS simulation results also varies between stations, indicating that the hydraulic characteristics of the cross-section — including width, bed depth, and channel roughness — play an important role in determining the pattern of water level rise at each point, not simply the magnitude of the discharge flowing through it.

Critical Segments and Implications for Flood Mitigation

The concentration of overflow points at stations 1 through 9, located in the downstream portion of the study segment near Singki' Bridge, indicates that the river cross-section in this area has relatively more limited conveyance capacity compared with the upstream portion (stations 10–15), which experiences overflow less frequently at lower return periods. This pattern is consistent with reports from local residents of recurring annual damage to public facilities, plantations, rice fields, and residential areas along the riverbanks, as well as bank erosion threatening nearby facilities. This critical segment should be a priority for river normalization planning, embankment reinforcement, or relocation of important facilities, given that even at a 2-year return period — a statistically frequent event — nine of the fifteen stations studied already experience overflow.

Consistency with Similar Studies

The pattern of increasing discharge and water level with increasing return period found in this study is consistent with other HEC-RAS-based flood simulation studies on rivers in South Sulawesi, such as the flood water level simulation of the Magala River in Sinjai Regency (Marsuki, 2021) and the Dangdo River in Tana Toraja Regency (Ikbal, 2021), which likewise reported water level increases in line with rising design discharge across various return periods. This agreement strengthens confidence that the combined approach of Gumbel-method rainfall frequency analysis, Nakayasu Synthetic Unit Hydrograph discharge calculation, and HEC-RAS hydraulic simulation is an appropriate and reliable methodology for evaluating flood risk in medium-sized watersheds such as the Saddang watershed.

Limitations of the Study

This study is subject to several limitations that should be considered in interpreting its results. First, the rainfall analysis relied on data from only one station (Makale Station), which does not capture the spatial variability of rainfall across the entire Saddang watershed, whose total area is far larger than the study segment. Second, the HEC-RAS simulation was carried out under one-dimensional steady flow conditions, which cannot capture the temporal dynamics of water level change or the two-dimensional lateral distribution of flow along the river cross-section. Third, the study segment was limited to only 2.5 km, so the simulation results

may not necessarily reflect the hydraulic behavior of the Saddang River as a whole, which has a total length of 38.3 km.

Recommendations for Further Research

To obtain a more detailed picture of flood dynamics over time, future studies are recommended to consider using an unsteady flow model that can capture temporal changes in discharge and water level, including flood arrival and recession times. In addition, modeling the flood flow pattern in two-dimensional (2D) format would provide a more realistic picture of inundation distribution across the entire affected area, particularly at the critical segment of stations 1 through 9 identified in this study. Expanding rainfall data coverage to additional stations and studying segments of the Saddang River beyond the boundaries of this research would also strengthen the generalizability of these findings for comprehensive management of the Saddang watershed.

Overall, the study establishes a clear chain from rainfall frequency analysis to design flood discharge and finally to hydraulic response. The Gumbel distribution is supported by the adopted goodness-of-fit tests; the Nakayasu method produces increasing peak discharges with increasing return period; and HEC-RAS translates these discharges into progressively higher water levels and a broader overflow pattern. The most persistent critical area is the downstream part of the modeled reach, particularly STA 1–9. This integrated result provides a useful preliminary basis for flood-risk identification and for prioritizing more detailed engineering assessment along the Saddang River.

Synthesis of Findings

The study reach is approximately 2.5 km, whereas the river length stated in the thesis is 38.3 km. Therefore, the results should be applied primarily to the Tagari–Singki reach and should not be generalized to the entire Saddang River without further investigation. Future research should expand rainfall observations to additional stations, extend the topographic and hydraulic model, and use unsteady-flow analysis. A 2D model would be particularly useful for converting overflow conditions into spatial inundation extents over settlements, agricultural areas, and infrastructure.

The results must be interpreted within the limits of the available hydrologic and hydraulic data. The rainfall frequency analysis uses one station, Makale Station, with ten years of annual maximum daily rainfall data. The complete record provides internal consistency, but a single station cannot fully represent spatial rainfall variability over a watershed. In addition, the HEC-RAS model uses steady-flow conditions. This approach is suitable for estimating water-surface profiles for specified design discharges, but it does not reproduce the time-dependent arrival, peak, and recession of an actual flood hydrograph.

Reliability and Limitations

The simulation is also useful as a screening and decision-support tool. Comparing multiple return-period profiles allows decision makers to distinguish sections that are consistently critical from those that become critical only at higher flows. The model results are consistent with the local observations reported in the thesis concerning recurring damage to public facilities, plantations, rice fields, residential areas, and riverbanks. This agreement increases the practical relevance of the findings, although the study does not include an independent numerical calibration or validation against observed flood levels.

The identified critical stations provide a practical basis for prioritizing detailed engineering assessment. Stations 1–9 should receive particular attention because they repeatedly show overflow in the modeled scenarios. The study indicates several possible management directions, including river normalization where appropriate, strengthening of vulnerable riverbanks, improvement of conveyance at constrained sections, and protection or relocation of important facilities exposed to recurrent flooding. Each measure should be evaluated using additional topographic, environmental, and structural information so that a local intervention does not transfer hydraulic effects to neighboring sections.

Implications for Flood Management

The return period should be understood as a statistical design concept rather than a fixed interval between floods. Q2, Q5, Q10, and Q25 represent different design conditions with different probabilities of exceedance. The fact that multiple stations overflow even for Q2 means that flood risk is not limited to rare extreme events. More frequent lower-return-period events can also produce substantial impacts when existing conveyance is insufficient.

The results establish a continuous relationship between the hydrologic and hydraulic components of the study. Design rainfall rises from 94.534 mm at Q2 to 161.990 mm at Q25, and peak discharge rises from 1052.427 m³/s to 1803.400 m³/s. At STA 1, the associated maximum flood depth rises from 7.37 m to 9.39 m. In relative terms, discharge increases by about 71.3%, whereas maximum flood depth increases by about 27.4%. The smaller percentage increase in depth reflects the nonlinear stage–discharge behavior of the natural cross section and does not indicate a proportionally small flood risk.

Relationship Between Return Period, Discharge, and Flood Depth

The energy slope also varies significantly along the reach. For Q2, the modeled values range from about 0.00160 at STA 1 to 0.01655 at STA 11, while Q25 ranges from about 0.00160 at STA 1 to 0.01600 at STA 15. This variation indicates that the hydraulic response is spatially heterogeneous. Therefore, flood-control measures should be designed from section-specific hydraulic information rather than applying a uniform intervention to the entire 2.5 km reach.

STA 1 is the downstream end of the modeled reach and consistently produces the maximum flood depth. Its riverbed elevation is 831.35 m. The computed water-surface elevation is 838.72 m for Q2 and 840.74 m for Q25, corresponding to depths of 7.37 m and 9.39 m. The low bed elevation combined with the computed high stage makes this section a key hydraulic control point in the study. Nevertheless, the actual inundation footprint depends on the surrounding overbank topography and should therefore be verified through field survey or two-dimensional mapping.

Hydraulic Significance of the Downstream Reach

As the return period increases, the affected area expands toward additional upstream sections. STA 4 becomes affected at Q5, while STA 11, STA 13, and STA 15 appear among the affected sections at Q10 and/or Q25; STA 10 is additionally affected at Q25. This progression suggests that the downstream critical zone is the first area to lose effective conveyance under increasing hydraulic loading, followed by a broader expansion of the flood-prone reach.

For mitigation planning, both the persistence and the expansion pattern should be considered.

The overflow pattern is persistent at stations 1, 2, 3, 5, 6, 7, 8, and 9 across all return periods. These stations form the most consistently affected part of the modeled reach. At Q2, nine stations are identified as overflowing, including STA 13. At Q5, ten stations are affected, while Q10 and Q25 produce progressively wider spatial impacts. The persistence of overflow at stations 1–9 is particularly important because it occurs even under the lowest design discharge examined.

Spatial Pattern of Overflow

At STA 6, one of the repeatedly affected sections, flood depth increases from 6.66 m at Q2 to 7.54 m at Q5, 8.08 m at Q10, and 8.68 m at Q25. At STA 9, the corresponding depths are 5.23, 6.07, 6.60, and 7.20 m. These values demonstrate that the critical downstream reach responds consistently to increasing discharge. The results also emphasize that a single maximum stage is insufficient to describe flood behavior; the longitudinal distribution of stage is needed to identify the sections where conveyance becomes inadequate.

The four HEC-RAS profiles show a clear upward shift of the water surface as the design discharge increases. At the downstream STA 1, the modeled flood depth is 7.37 m for Q2, 8.22 m for Q5, 8.77 m for Q10, and 9.39 m for Q25. The maximum depth therefore increases by 2.02 m between Q2 and Q25. This increase is hydraulically meaningful because the simulations also show that the water surface extends above the available bank condition at several stations, producing overflow. The stage response is not perfectly linear because natural river geometry changes from one cross section to another.

Comparative Analysis of Water Levels

Table 12. Flood depth at representative stations for four return periods

Station	Riverbed Elevation (m)	Water Level Q2 (m)	Water Level Q5 (m)	Water Level Q10 (m)	Water Level Q25 (m)
1	831.35	7.37	8.22	8.77	9.39
3	832.27	6.62	7.46	8.01	8.63
6	832.92	6.66	7.54	8.08	8.68
9	835.65	5.23	6.07	6.60	7.20
15	842.00	5.14	5.74	6.16	6.61

Table 12 highlights the contrast between the upstream and downstream sections. STA 1 and STA 6 show the largest depths among the representative stations, while STA 15 remains lower despite the larger upstream bed elevation. The table also confirms that every representative station experiences an increase in modeled depth as the return period increases. This consistency supports the interpretation that the increase in design discharge is translated into a systematic rise in hydraulic stage throughout the modeled reach, with the magnitude controlled by local channel geometry.

The central finding of the study is that the Saddang River reach between Tagari and Singki has limited hydraulic capacity at several locations under the modeled design conditions. The

problem is most persistent in the downstream portion, where STA 1–9 repeatedly experience overflow and STA 1 produces the maximum modeled water depth. Increasing the return period increases both the hydraulic loading and the spatial extent of affected stations. The results therefore provide a clear basis for prioritizing the downstream reach for field verification and flood-management planning.

Overall Discussion

The study also demonstrates the value of combining hydrologic and hydraulic analyses. Hydrologic analysis alone provides the magnitude of the design discharge, but it does not indicate where the river may overflow. Conversely, a hydraulic model without defensible design-flow inputs cannot adequately represent the severity of a design event. In this study, the Gumbel frequency analysis, Nakayasu synthetic hydrograph, and HEC-RAS simulation form a sequential analytical framework. The resulting information can therefore support preliminary identification of critical sections before more detailed unsteady-flow, two-dimensional, or field-validated modeling is undertaken.

The results can be used to establish priorities for subsequent engineering investigation, but they should not by themselves be interpreted as a final design for flood-control structures. The first priority is to verify the geometry and bank elevations at the repeatedly overflowing stations through detailed field survey. The second priority is to evaluate whether local conveyance can be improved without creating adverse hydraulic effects upstream or downstream. The third priority is to assess the exposure of settlements, agricultural land, public facilities, and other assets in the affected area. These steps would allow the modeled water levels to be translated into an engineering risk-management program.

Engineering Implications of the Simulation

The expansion of affected stations at higher return periods provides a second dimension of risk. At Q5, the affected pattern includes STA 4, while the Q10 and Q25 scenarios extend the modeled impact to additional upstream stations. This means that the flood problem can be viewed as a core critical zone surrounded by an expanding area of hydraulic influence. The downstream stations 1–9 therefore represent a priority for detailed investigation, while stations that become affected only at larger return periods should also be considered in long-term flood management planning.

A particularly important finding is the persistence of overflow at STA 1, 2, 3, 5, 6, 7, 8, and 9 across all four return-period scenarios. This persistence indicates that these sections are not merely sensitive to the largest simulated flood. Instead, they repeatedly show insufficient conveyance under the modeled conditions. The presence of overflow at the Q2 scenario is especially relevant for local risk management because a lower return period represents a more frequent design condition than Q10 or Q25. Consequently, mitigation focused only on extreme events could leave the community exposed to recurrent flooding under lower-magnitude conditions.

Flood-Risk Meaning of Persistent Overflow

The energy-slope results support this interpretation. At Q2, the energy slope is 0.016548 at STA 11 but only 0.001601 at STA 1. At Q25, the corresponding values are 0.013781 and 0.001601. The reduction in energy slope toward the downstream end is accompanied by a high modeled water depth at STA 1. This does

not mean that the energy slope alone causes flooding; rather, it demonstrates that the hydraulic state at each station is the result of the combined effects of discharge, channel geometry, elevation, and roughness. The HEC-RAS profile should therefore be interpreted as a spatially varying hydraulic system.

The station results demonstrate that water depth is strongly influenced by the local riverbed profile. For example, the riverbed elevation at STA 15 is 842.00 m and decreases generally toward STA 1, where it reaches 831.35 m. At Q2, the water depth varies from 4.02 m at STA 11 to 7.37 m at STA 1. At Q25, the range is from 5.60 m at STA 11 to 9.39 m at STA 1. Thus, two stations carrying the same total discharge can have substantially different modeled water depths because their cross-sectional geometry and bed elevations are different.

Section-by-Section Hydraulic Response

The calculated peak discharge sequence also shows that the increase is not limited to the Q25 extreme case. The discharge rises from 1052.427 m³/s at Q2 to 1343.124 m³/s at Q5, an increase of 290.697 m³/s. It then rises by 208.809 m³/s to Q10 and by a further 251.467 m³/s to Q25. These changes demonstrate why several return periods are useful in hydraulic assessment. A single design discharge would not reveal how the extent of potential overflow changes as hydraulic loading becomes progressively more severe.

The Nakayasu hydrographs provide the transition between rainfall analysis and hydraulic simulation. The peak discharge for every return period occurs at approximately the seventh hour, while the magnitude of the peak changes according to the return-period rainfall. This common peak timing suggests that the characteristic response time of the modeled watershed remains unchanged between scenarios; what changes is primarily the magnitude of runoff generated by the different effective rainfall inputs. In practical terms, the model indicates that increasing rainfall severity does not alter the adopted watershed response parameters, but it increases the amount of water that must be conveyed through the river channel.

Interpretation of the Nakayasu Hydrograph Response

The Gumbel design values increase from 94.534 mm for the 2-year return period to 161.990 mm for the 25-year return period. The increase of 67.456 mm represents approximately 71.4% of the Q2 design rainfall. The close correspondence between this increase and the approximately 71.3% increase in the calculated peak discharge indicates that the hydrologic model transfers the increase in design rainfall into a substantially larger flood response. The goodness-of-fit results provide additional support for using the selected Gumbel distribution within the scope of the available data. Nevertheless, the relatively short ten-year rainfall record means that the higher-return-period estimates should be treated as design estimates subject to hydrologic uncertainty rather than as exact predictions of future rainfall.

The rainfall frequency results provide an important basis for understanding the magnitude of the hydraulic scenarios simulated in HEC-RAS. The annual maximum daily rainfall data contain a relatively wide range, from 71 mm in 2020 to 145 mm in 2019. The mean value of 97.7 mm and standard deviation of 22.58 mm indicate substantial year-to-year variation within the ten-year record. The positive skewness coefficient of 0.781 also indicates that several higher rainfall observations contribute to the asymmetric distribution of the series. This characteristic is relevant

because flood-producing rainfall is generally associated with the upper portion of the observed frequency distribution rather than with the mean condition.

Interpretation of the Design Rainfall Results

Table 13. Integrated hydrologic–hydraulic summary

Station	Riverbed Elevation (m)	Water Level Q2 (m)	Water Level Q5 (m)	Water Level Q10 (m)	Water Level Q25 (m)
1	831.35	7.37	8.22	8.77	9.39
3	832.27	6.62	7.46	8.01	8.63
6	832.92	6.66	7.54	8.08	8.68
9	835.65	5.23	6.07	6.60	7.20
15	842.00	5.14	5.74	6.16	6.61

CONCLUSION

Based on the analysis and discussion, it can be concluded that the calculation of design flood discharge using the Nakayasu Synthetic Unit Hydrograph method shows discharge increasing with increasing return period, namely 1052.427 m³/s (Q2), 1343.124 m³/s (Q5), 1551.933 m³/s (Q10), and 1803.400 m³/s (Q25). This shows that the larger the return period, the greater the potential peak discharge that must be accommodated by the Saddang River cross-section.

The flood simulation results using HEC-RAS show that the increase in design discharge directly affects the rise in water level and the number of stations experiencing overflow. At larger return periods, inundation occurs at more stations, with the maximum water level continuing to rise, from 7.37 m at Q2 to 9.39 m at Q25. This condition indicates that in several segments — particularly stations 1 through 9 — the capacity of the Saddang River cross-section is insufficient to accommodate the design flood discharge, potentially causing flooding in the surrounding area.

In general, this study shows that the Saddang River has an increasing flood risk potential as the return period increases, so that targeted flood management and control measures are needed in the segments prone to overflow.

Based on the results of this study, it is recommended that future studies consider the use of an unsteady flow model so that flood dynamics can be analyzed in greater detail over time. In addition, it is also recommended to model the flood flow pattern in two-dimensional (2D) format to obtain a more realistic picture of flow distribution across the entire affected area.

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