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# Hydrological and Hydraulic Assessment for Optimizing Flood Mitigation: Sringin Watershed Case Study, Semarang, Indonesia

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**Abstract**. Flooding is a recurring hazard across Indonesia, particularly in urban regions such as Semarang City, where high-intensity rainfall, tidal surges, land subsidence, and changes in land use contribute to frequent inundation. This study aims to develop an integrated hydrological and hydraulic model for flood control in the Sringin River. The hydrological analysis method, namely rainfall to runoff, is carried out using the Nakayasu synthetic unit hydrograph. The hydrological component involves calculating design rainfall for various return periods. Design rainfall is used as input for hydraulic analysis using HEC-RAS software to obtain the output of the Sringin River water level at return periods of 2 years, 5 years, 20 years, 25 years, and 50 years. For a 25-year return period, the main river channel produced a simulated flood discharge of 47.42 $m^3/s$, resulting in water overtopping the left embankment by 0.545 m at station P.30. Similar overflow conditions were observed at multiple stations, including P.1, P.1A, P.3A, P.5A, P.28, P.28A, P.29, and P.30. In the tributary segment (Sta A1 to A15), a design discharge of 49.80 $m^3/s$ also led to overtopping by 0.545 m. These results highlight a significant discharge deficit between the calculated flood flow and the existing capacity of the river cross-sections.

**Keywords**: Bankfull Capacity, Flood mitigation, HEC-RAS simulation, Nakayasu, Sringin Watershed

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## 1. Introduction

Semarang is capital of Central Java, one of cities in the region that frequently affected by flood and tidal inundations[1,2]. Areas along its northern shore—including Genuk, Kaligawe, Tambakrejo, Kemijen, Karangayu, Tawang Station, and A. Yani Airport—are particularly vulnerable[3–10]. These recurring events damage public and private infrastructure, especially residential buildings [11]. Furthermore, they cause significant economic disruption and traffic bottlenecks, primarily along stretches of the National Road.

The government has constructed various flood prevention infrastructures along Semarang's northern shore, including retention ponds with pump stations. Key projects include systems along the Sringin, Grace, Banger, and Semarang Rivers, as well as the Tawang Polder [12]. Specific measures such as the long storage channels on the Sringin, Banger, and Tenggang Rivers ("longstarage kali") have also been implemented.

The primary function of a retention pond is to temporarily store water during a river's peak discharge and release it slowly once the water level recedes. However, flooding remains a persistent issue to high rainfall intensity. According to disaster data, 88 flood events still account for the majority of incidents in Semarang City, each typically causing inundations of 30-70 cm. These events are attributed to high rainfall intensity and inadequate drainage capacity [13,14].

Therefore, the objectives of this study are to calculate the anticipated flood discharge in the Sringin watershed and to identify effective flood prevention strategies for Semarang.

## 2. Methods

This study focuses on the Sringin Watershed. The Sringin Watershed is located in East Semarang District, Semarang City. Administratively, it borders Demak Regency. Figure 3 shows a map of the Sringin Watershed. [1,10].

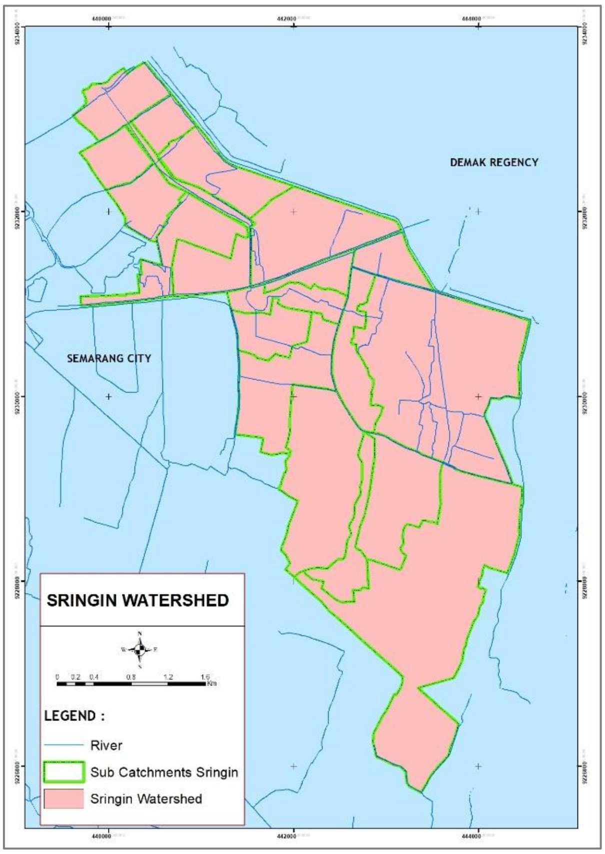


**Figure 1.** Sringin Watershed Location

*2.1. Hydrological and Hydraulic Analysis Methods*

A hydrological study serves as a fundamental prerequisite in the design of hydraulic structures, as it provides essential parameters such as flood discharge, duration, and peak time**,** which are required for subsequent stages of analysis. The hydraulic analysis then constitutes the next critical phase, in which the principles of fluid mechanics are applied to model the movement of water through engineered systems such as open channels, pipelines, and retention ponds. Consequently, the data obtained from the hydrological analysis are translated into practical design specifications[15,16].

Specifically, the hydraulic analysis relies heavily on the outcomes of the hydrological study, particularly on the difference between the calculated flood discharge for a given return period and the existing flow capacity of the river cross-section. This calculation is closely related to the time of concentration and flood peak characteristics. During periods of intense rainfall across the watershed, the flow increases from normal to peak conditions[17–19].

In the process of designing hydraulic structures, the hydrological analysis generally represents the initial stage, providing a quantitative basis for subsequent hydraulic evaluations. The following steps are typically undertaken to determine the design discharge [20,21]:

a. Determine the area of the watershed (catchment).
b. Define the influence area of each rainfall station.
c. Use the available rainfall data to calculate the average maximum daily rainfall of the watershed.
d. Determine the design rainfall corresponding to a specific return period (T years).
e. Calculate the design flood discharge based on the design rainfall for the return period *T*.

## 3. Results and Discussion

The annual daily rainfall calculation for this research location uses data spanning 10 years, from 2010 to 2020, and is expressed as yearly rainfall. Only 1 (one) rain station, the Karangroto rainfall station, which is the closest to the watershed, was used in this study.This information is derived from the maximum daily rainfall data at the Karangroto Rain Station in order to examine the hydrology, particularly the determination of the maximum average rainfall.The Karangroto Rain Station is the only factor affecting the Sringin Watershed's catchment area .Therefore, the Sringin Catchment Area's coefficient/weight is 1.

The Karangroto Rain Station, which is also the Sringin watershed, has a maximum rainfall table that can be found in Table 1.

**Table 1.** Maximum Rainfall at Karangroto Rain Station

| Year | date | Maximum Rainfall Based Karangroto Station (mm) |
|---|---|---|
| **2011** | 02-Jan | 100 |
| **2012** | 04-Feb | 182 |
| **2013** | 23-Feb | 135 |
| **2014** | 23-Jan | 135 |
| **2015** | 13-Feb | 130 |
| **2016** | 27-Dec | 110 |
| **2017** | 20-Jan | 110 |
| **2018** | 09-Mar | 85 |
| **2019** | 04-Apr | 116 |
| **2020** | 20-Feb | 95 |

Source: Analysis Results, 2025

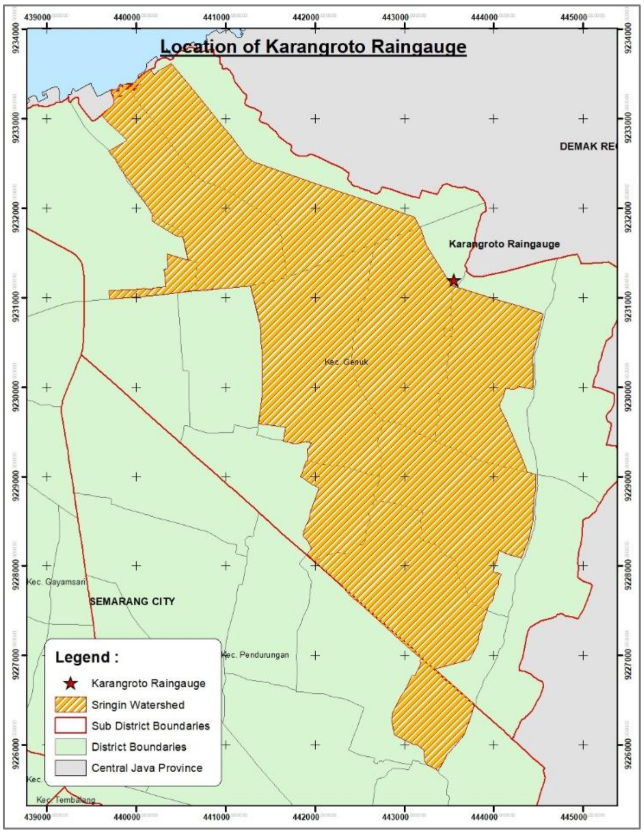

**Figure 2**. Map of the Karangroto Rainfall Station.

To calculate the rainfall return period, four probability distributions were evaluated: Normal, Log-Normal, Gumbel, and Log-Pearson Type III [5]. The optimal distribution was selected based on an initial analysis of statistical parameters, followed by validation with the Chi-Square and Smirnov-Kolmogorov goodness-of-fit tests. The results of this distribution selection process are provided in Table 2 and Table 3.

**Table 2.** Distribution selection based on condition

| No | Distribution | Condition | Result | Remark |
|---|---|---|---|---|
| 1 | Normal | $Cs \approx 0 \pm 0{,}3$ | 0.201 | fitted |
| | | $Ck \approx 3$ | 3.288 | Fitted |
| 2 | Log Normal | $Cs \approx Cv^3 + 3Cv$ | -0.293 | Fitted |
| | | $Ck \approx Cv^8 + 6Cv^6 + 15Cv^4 + 16Cv^2 + 3$ | 3.240 | Fitted |
| 3 | Gumbel | $Cs \leq 1.14$ | 0.201 | Fitted |
| | | $Ck \leq 5.4$ | 3.288 | Fitted |
| 4 | Log Person III | If not fitted | -0.293 | Fitted |
| | | | 3.240 | Fitted |

Source: Analysis Result, 2023

Following the determination of design rainfall, the Synthetic Unit Hydrograph (HSS) Nakayasu method was utilized to calculate the corresponding flood discharge for each return period [22]. This step is essential for hydraulic analysis, as it determines the magnitude of the flood event that the hydrologic structures must be designed to handle. The following are the parameters of the Sringin watershed as input to the Nakayasu model

.

**I. Characteristics of Watersheds and Rainfall**

| | | |
|---|---|---|
| Name DAS/River | | |
| A = | 13,6 | $km^2$ |
| L = | 6,3 | km |
| R = | 1 | mm |
| Tr= | 1 | jam |

**II. Synthetic unit hydrograph parameters :**

| | | | | |
|---|---|---|---|---|
| $T_g$ | $0.21*L^{0.7}$ | < 15 km | | O'clock |
| | 0.4 + 0.058*L | > 15 km | 1,278 | |
| Tr | 0.75*Tg | = | 0,959 | O'clock |
| $T_{0.8}$ | 0.8*Tr | = | 0,767 | O'clock |
| $T_p$ | Tg+0.8*Tr | = | 2,046 | O'clock |
| a | (0.47(A.L)^0.25) / Tg | = | 1,777 | |
| $T_{0.3}$ | a*Tg | = | 2,271 | O'clock |
| | $T_p+T_{0.3}$ | = | 4,317 | O'clock |
| | $T_p+T_{0.3}+1.5*T_{0.3}$ | = | 7,724 | O'clock |
| Qp | | = | 3,467 | $m^3/s$ |

**III. Check Volume and Height of Overflow**

| | | | |
|---|---|---|---|
| Volume Rain | = | 36,011 | $m^3$ |
| Volume HSS | = | 54,266 | $m^3$ |
| DRO | = | 1,507 | mm |

To determine whether the model being used in nakayasu modeling is acceptable for the field conditions, model verification step is required. number of statistical measures, such as the Nash-Sutcliffe Efficiency (NSE) number, Percent BIAS, and the Root Square Mean Equation (RMSE) were used to test the verification results [23,24]. The statistical outcomes are displayed in Figure 4. The calculation value is better in the empirical method when the RMSE and BIAS values are smaller, while NSE is near to 1 (one) [25,26].

**Table 3.** Nakayasu Model Verification Statistics

| Parameters Statistic | The statistical Outcomes |
|---|---|
| RMSE | 0.3 |
| Percent BIAS | 0.59% |
| NSE | 0.933 |

The results of these flood discharge calculations for the 2, 5, 10, 25, and 50-year return periods are presented in Table 5.

**Table 4.** Flood discharge for Many Return Periods

| Return Period (year) | Discharge ($m^3$/dt) |
|---|---|
| 2 | 35.51 |
| 5 | 41.37 |
| 10 | 44.15 |
| 25 | 47.42 |
| 50 | 49.80 |

The flood water level of the Sringin River was simulated using HEC-RAS version 5.0.7. The model geometry was constructed using data from a long cross-section and a situation map of the river is presented in Figure 3. The simulation of flood levels required inputs of the Manning's roughness coefficient and the flood discharge values for various return periods, as calculated previously. Manning coefficient is shown in Figure 4.

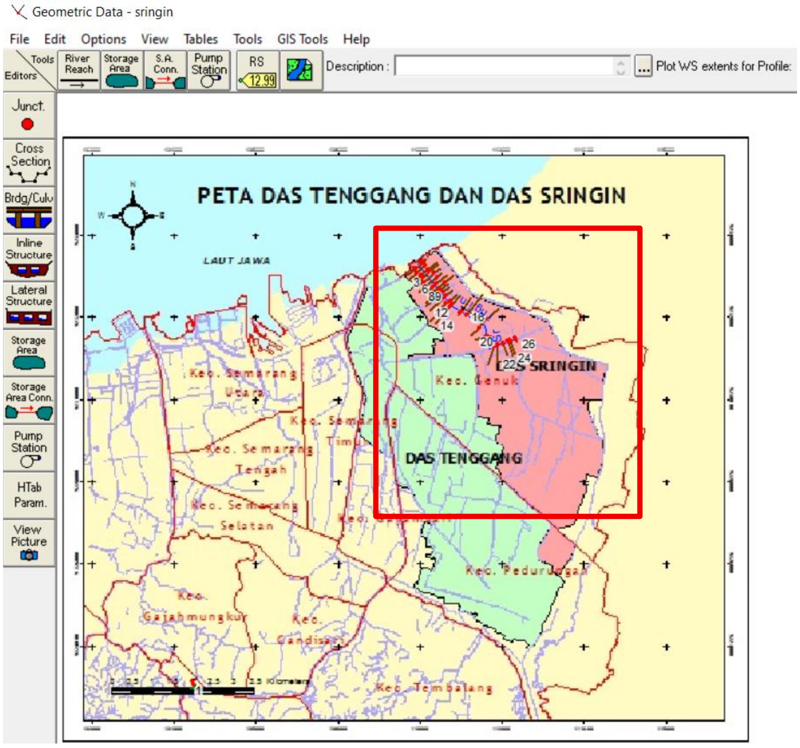


**Figure 3.** Sringin River Geometry

Edit Manning's n or k Values

River: Sringin | Reach: 1 | All Regions | Edit Interpolated XS's | Channel n Values have a light green background

Selected Area Edit Options: Add Constant ... | Multiply Factor ... | Set Values ... | Replace ... | Reduce to L Ch R ...

| | River Station | Frctn (n/K) | n #1 | n #2 | n #3 |
|---|---|---|---|---|---|
| 1 | 26 | n | 0.025 | 0.013 | 0.025 |
| 2 | 25 | n | 0.025 | 0.013 | 0.025 |
| 3 | 24 | n | 0.025 | 0.013 | 0.025 |
| 4 | 23 | n | 0.025 | 0.013 | 0.025 |
| 5 | 22 | n | 0.025 | 0.013 | 0.025 |
| 6 | 21 | n | 0.025 | 0.013 | 0.025 |
| 7 | 20 | n | 0.025 | 0.013 | 0.025 |
| 8 | 19 | n | 0.025 | 0.013 | 0.025 |
| 9 | 16 | n | 0.025 | 0.013 | 0.025 |
| 10 | 15 | n | 0.025 | 0.013 | 0.025 |
| 11 | 14 | n | 0.025 | 0.013 | 0.025 |
| 12 | 13 | n | 0.025 | 0.013 | 0.025 |
| 13 | 12 | n | 0.025 | 0.013 | 0.025 |
| 14 | 11 | n | 0.025 | 0.013 | 0.025 |
| 15 | 10 | n | 0.025 | 0.013 | 0.025 |
| 16 | 9 | n | 0.025 | 0.013 | 0.025 |
| 17 | 8 | n | 0.025 | 0.013 | 0.025 |
| 18 | 7 | n | 0.025 | 0.013 | 0.025 |
| 19 | 6 | n | 0.025 | 0.013 | 0.025 |
| 20 | 5 | n | 0.025 | 0.013 | 0.025 |
| 21 | 4 | n | 0.025 | 0.013 | 0.025 |
| 22 | 3 | n | 0.025 | 0.013 | 0.025 |
| 23 | 2 | n | 0.025 | 0.013 | 0.025 |
| 24 | 1 | n | 0.025 | 0.013 | 0.025 |

**Figure 4.** Manning Coefficient

In Figure 4, the Manning coefficients on the riverbed and the right and left embankments are different. The riverbed consists of concrete material with a trowel finish so that the manning price is 0.013. The river's right and left embankments consist of the same material, namely gravel which has a manning value of 0.025. Small Manning indicates a smoother or slippery surface, which means the flow will be faster. A larger Manning means the surface is rougher, so there is resistance to the flow of discharge A larger manning means a rougher surface so there is resistance to the discharge flow.

The HEC-RAS analysis was conducted for return periods of 2, 5, 10, and 25 years along a section of the Sringin River comprising 45 stations (Sta) on the main channel (P) and 15 stations on a tributary (A).

**Table 4.** Planned flood discharge with HSS Nakayasu Method

| Return Period (year) | Water Surface Elevation (m) |
|---|---|
| 2 | 1.45 |
| 5 | 1.66 |
| 10 | 1.81 |
| 25 | 2.24 |
| 50 | 2.27 |

The key results are as follows:

a. **2-year return period**: Eight stations experienced overflow (Sta P.1, P.1A, P.3A, P.5A, P.28, P.28A, P.29, P.30). The maximum flood depth of 0.465 m was recorded on the left embankment at Sta P.1.
b. **5-year return period:** Nine stations experienced overflow (the same eight as above, plus Sta P.27). The maximum flood depth remained 0.465 m on the left embankment at Sta P.1.
c. **10-year return period**: Nine stations experienced overflow (identical to the 5-year return period). The maximum flood depth increased to 0.486 m on the left embankment at Sta P.30.
d. **25-year return period**: Nine stations experienced overflow (identical to the 10-year return period). The maximum flood depth further increased to 0.545 m on the left embankment at Sta P.30.

The results indicate that a consistent set of nine stations (Sta P.1, P.1A, P.3A, P.5A, P.27, P.28, P.28A, P.29, P.30) are prone to overflow for all return periods of 5 years and greater. Furthermore, the maximum flood depth increases with the return period and its location shifts from Sta P.1 (for 2 and 5-year events) to Sta P.30 (for 10 and 25-year events).

These findings are critical for assessing the impact of floods of various severities on the Sringin River system. They provide a vital evidence base for informing decisions on flood management strategies, infrastructure design, and emergency preparedness planning in the region. A longitudinal profile of the Sringin River channel, resulting from the cross-sectional modeling, is presented in Figure 3.

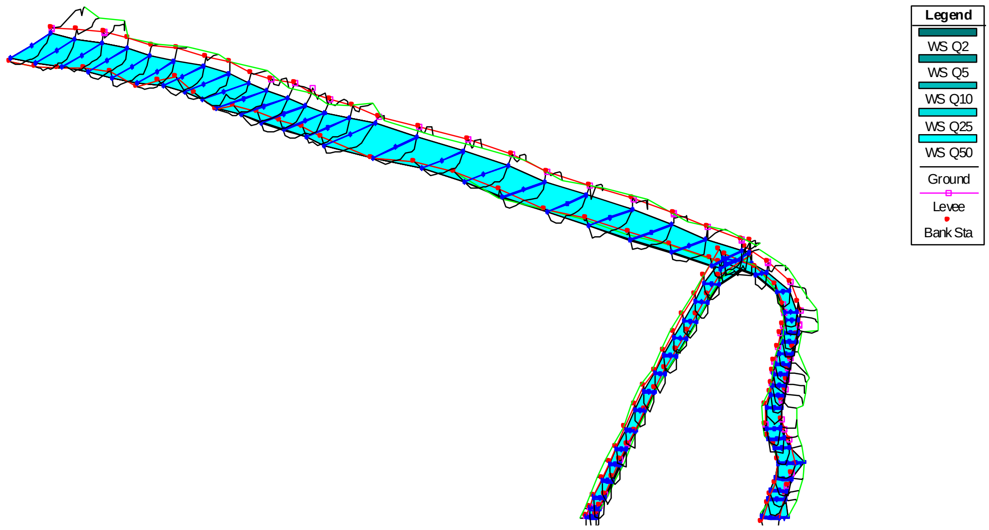


**Figure 4**. Longitudinal profile of the Sringin River channel

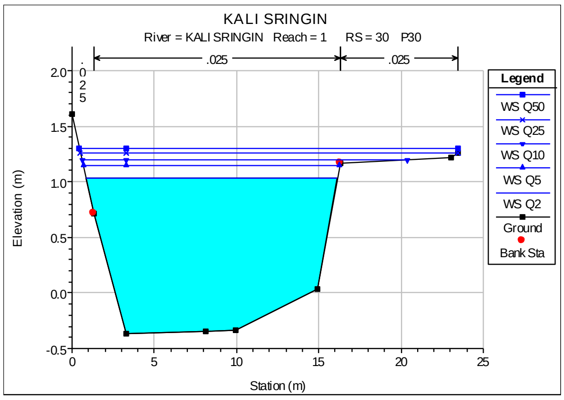


**Figure 4**. Cross section P.30

The HEC-RAS simulation for a 25-year return period flood, shown in Figure 4, indicates that the water level at cross-section P.30 will exceed the left embankment by 0.545 m. This overflow is localized to the left side, as the right embankment level is not exceeded.

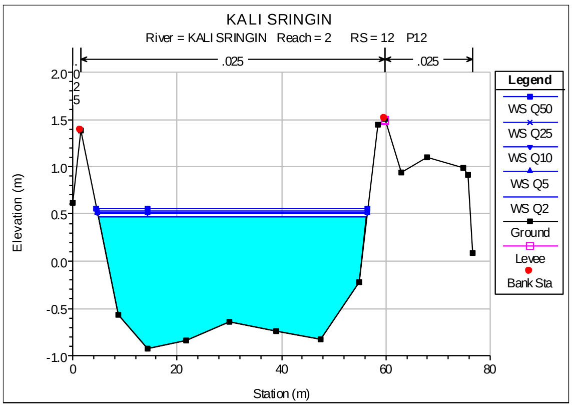


**Figure 5.** Cross section of the P.12 creek

For the 25-year return period flood, HEC-RAS analysis (Figure 6) indicates that the tributary cross-section at P.12 will experience embankment overtopping, with a 0.545 m overflow on the left side and additional overflow on the right embankment.

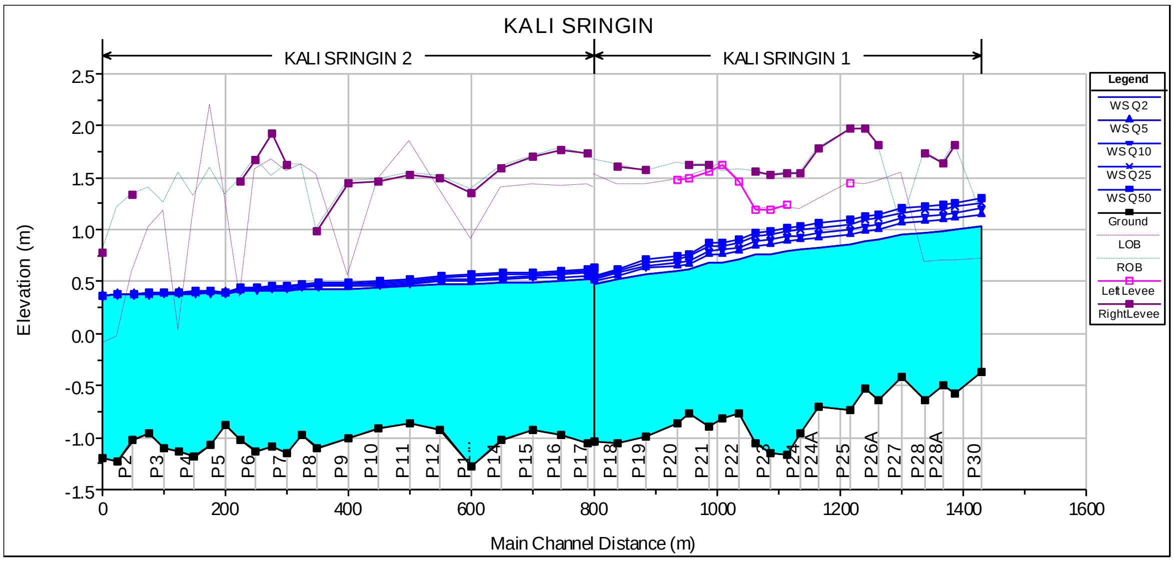


**Figure 6.** Cross section Kali Sringin Main River

The longitudinal profile of the Sringin River (Sta P.2 to P.30), analyzed for a 25-year return period using HEC-RAS (Figure 5), indicates water overtopping the embankments along its entire length. The magnitude of overflow increases significantly from 0.25 m at Sta P.2 to 0.588 m within the reach between Sta P.18 and P.30. The corresponding hydraulic behavior of the river's tributaries is presented separately in Figure 7.

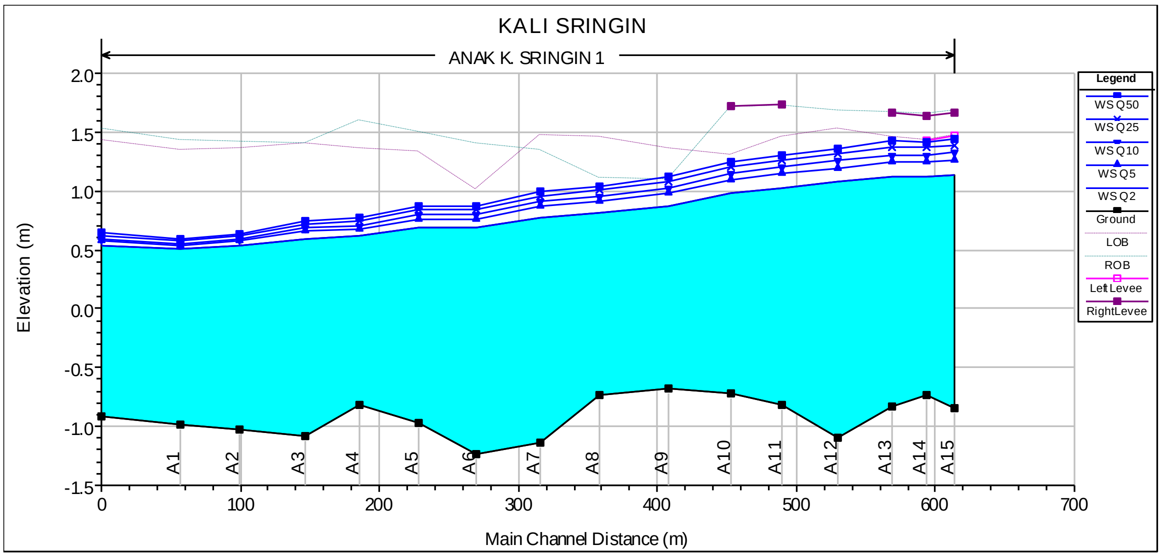


**Figure 7.** Cross section of Kali Sringin tributary

As illustrated in Figure 7, the HEC-RAS analysis for the 50-year return period flood on the Sringin tributary (Sta A.1 to A.15) shows a maximum water level reaching 0.545 m.

Based on these results, it is evident that peak rainfall events generate river discharges capable of producing water levels exceeding the current channel capacity. To mitigate overflow flooding, two primary solutions are recommended:

a. Raise the existing embankments within the polder system to a height greater than the projected 0.545 m water level to contain the 50-year flood event.
b. Construct a retention pond or long storage basin upstream to reduce the peak discharge entering the tributary, thereby lowering the water level during extreme rainfall.

## 4. Conclusion

This investigation computed design rainfall values for various return periods. The maximum rainfall intensity for the 10-year return period was found to be 113 mm. Using these values, flood discharges were simulated with HEC-RAS. The key results are as follows:

a. For a 25-year return period, the simulated flood discharge is 47.42 $m^3/s$, resulting in water overtopping the left embankment by 0.545 m at station P.30. This overflow condition also occurred at multiple other stations along the main river (P.1, P.1A, P.3A, P.5A, P.28, P.28A, P.29, P.30).
b. For the tributaries (Sta A1 to A15), a 25-year return period flood with a design discharge of 49.80 $m^3/s$ also resulted in a water level exceeding the embankment by 0.545 m.


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