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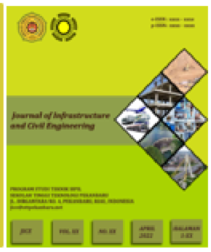

Scenario-Based Analysis of Hydraulic Response in the Senapelan River Reach

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Abstract

The absence of long-term observed hydrological data poses a significant challenge for urban flood management. This study addresses data scarcity in the Kedungsari reach of the Senapelan River, Pekanbaru, Indonesia, by combining 1D steady-flow hydraulic modeling with empirical hydrological estimation to determine the channel's critical bankfull capacity and its corresponding return period. The model isolated the maximum safe bankfull capacity at 17.20 m^3/s, identifying Cross-Section 4 as the primary hydraulic bottleneck. Mapping this physical capacity onto the regional design flood frequency curve revealed that the existing channel corresponds to a low return period of approximately 1.5 years. Consequently, the channel is severely undersized, failing to contain a standard 2-year frequent flood (24.70 m^3/s) or the legally mandated 10-year extreme event (33.26 m^3/s).

Keywords: Bankfull capacity, Data scarcity, HEC-RAS, Ungauged river

1. Introduction

One of the common problems in urban areas is flooding. Rapid urbanization, population growth, and changes in land use have led to a decrease in water absorption. As a result, river flow rates have increased, often causing flooding along riverbanks. The Senapelan river, is highly prone to flooding, due to its frequent of flood in a particular river reach. Heavy rain over a few hours has caused flooding in several areas of the Senapelan river reach. This flood causing significant impact on the local activity and fast-occurring floods can result in substantial damage [1]. The lack of long-term, reliable discharge monitoring data for this river presents a significant challenge for the conventional application of flood frequency analysis.

Despite the recurring issue of flooding, the absence of observed hydrological data presents a significant challenge in accurately quantifying the specific flood magnitudes responsible for inundation, as well as in understanding the associated hydraulic characteristics, such as depth and velocity, at different flood stages [2][3]. This lack of data impedes the ability to assess the river's capacity, identify critical flow thresholds, and predict the extent of flooding under various hypothetical scenarios [4]. Such limitations not only hinder flood forecasting but also restrict the development of more effective mitigation strategies and the planning of safer infrastructure. In this context, the collection of more

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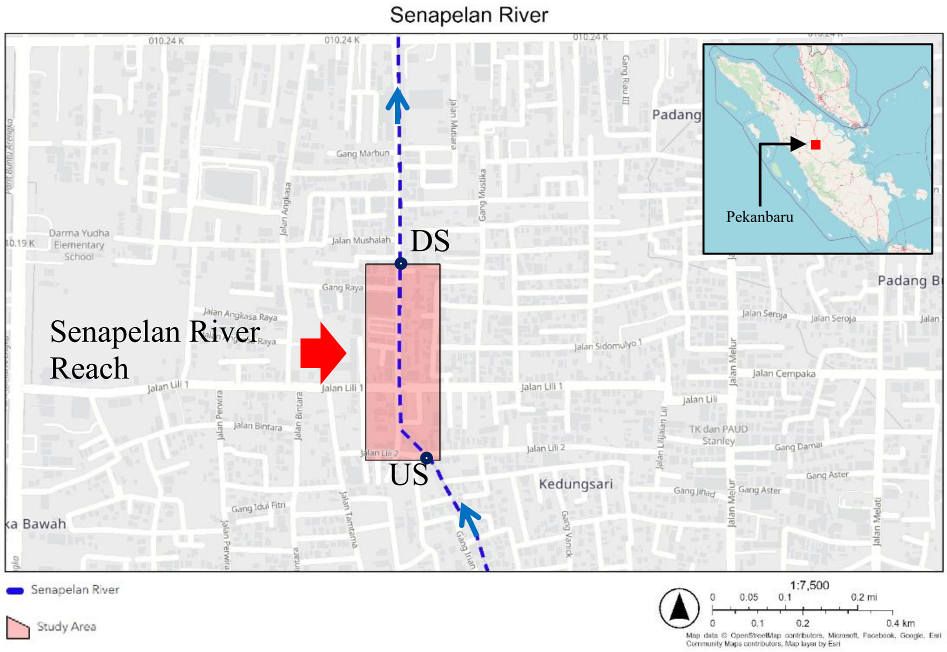
comprehensive and reliable data becomes crucial, providing a stronger foundation for future flood risk analysis and management.

To overcome the data scarcity, this study employs an inverse scenario-based hydraulic modeling approach paired with empirical hydrological estimation. Hydraulic modeling analysis is essential to precisely determine water surface profiles under different flow conditions [5]. This study aims to precisely identify the critical bankfull capacity threshold of the river reach and determine its corresponding return period under existing conditions. By applying a step-incremental range of hypothetical discharge magnitudes to the geometric model, the river's physical capacity is systematically investigated [6]. This capacity is then mapped against a regional design flood frequency curve derived from rainfall data. This framework allows for a proactive understanding of flood vulnerability in data-scarce urban catchments without relying on historical flow records.

2. Methodology

2.1 Study Area

This research was conducted on the Senapelan River reach, which is located in Kedungsari, Sukajadi District, Pekanbaru City, Riau Province, Indonesia. This reach spans 413 meters, from the upstream point at 0°31'36.7"N, 101°25'41.9"E to the downstream point at 0°31'49.7"N, 101°25'40.0"E (Fig. 1).



Source : Open Street Map, ESRI Map Layer (2025)

Figure 1. Study Area

2.2 Data Collection & Preparation

The data used in this study consists of channel geometry data. The channel geometry was obtained through field measurement surveys and consists of river cross sections, where the floodplain is separated by river embankment points. These river cross sections were provided by the Sumatra III River Basin Agency (*Balai Wilayah Sungai Sumatera III*). The Manning roughness coefficient is set to 0.02, based on the channel conditions described in the HEC-RAS Hydraulic Reference Manual [8]. This value reflects the characteristics of the channel, which has a concrete bottom and sides constructed with plastered cement rubble masonry. At the downstream cross section, the observed water level elevation is 5.16 m.

For the hydrological aspect, the catchment area and upstream river length were derived from a digital elevation model called DEMNAS and satellite imagery. DEMNAS data was obtained from the Indonesian Geospatial Agency (*Badan Informasi Geospasial*, BIG). The river path was first digitized using Google Earth Pro with the help of satellite imagery and field observations. This digitized river path was then measured to obtain the reach length. The rainfall data used in this study is obtained from [7], which provides values based on return periods.

2.3 Research Framework and Evaluation Benchmarks

A dual-stage framework was developed to address data scarcity and characterize the flood hazard potential of the study area. First, an incremental steady-flow simulation was executed in HEC-RAS to determine the true physical bankfull capacity. Discharges were adjusted via trial and error until overbank flow was initiated at the most restrictive cross-section of the reach.

Secondly, to contextualize this physical capacity within local risk standards, a hydrological analysis was conducted to establish benchmarks for frequent and extreme design floods. These benchmarks serve as a comparative baseline to determine the statistical return period of the channel's existing capacity.

1. Frequent flood benchmark: Represented by the 2-year return period design flood (Q2), reflecting typical frequent runoff events.
2. Extreme Flood Benchmark: Represented by the 10-year return period design flood (Q10) as specified in Regulation No. 12/PRT/M/2014 of the Minister of Public Works for the management of urban drainage in metropolitan areas.

The two design flood benchmarks used in this study to contextualize the bankfull capacity are derived from statistical return periods. A hydrological analysis was conducted to estimate these design flood magnitudes using the empirical Rational Method equation:

$$Q = \frac{C\ I\ A}{360} \tag{1}$$

where :

Q = design flood (m^3/s);
I = rain intensity (mm/hour);
A = catchment area (ha);
C = runoff coefficient.

Rainfall intensity (I) is calculated using the Mononobe method, as shown in the following equation:

$$I = \left(\frac{R}{24}\right)\left(\frac{24}{tc}\right)^{\frac{2}{3}} \tag{2}$$

where :

R = daily rainfall (mm);
tc = time of concentration (hour).

Time of concentration (tc) is calculated using the Kirpich method, as shown in the following equation:

$$tc = 0.0195\ L^{0.77} S^{-0.385} \tag{3}$$

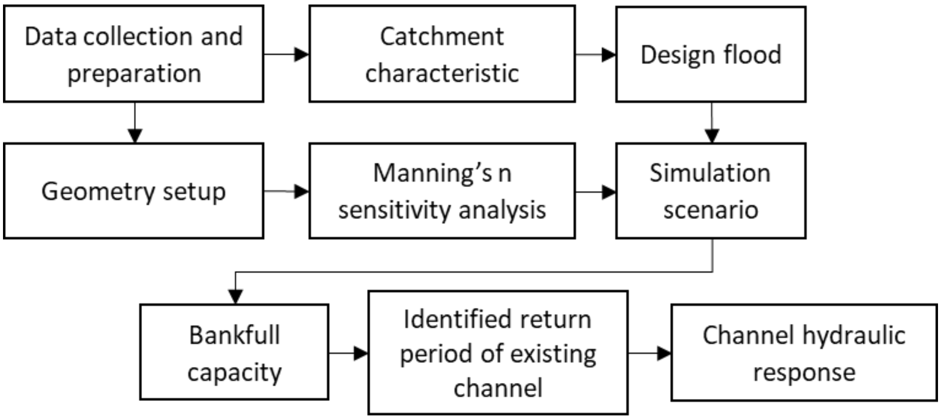
where :

L = river reach length of catchment area (m);
S = river reach slope of catchment area.

Based on the discussion above, the research flowchart has been developed and is presented in the figure 2.

2.4 HEC-RAS

HEC-RAS (Hydrologic Engineering Center's River Analysis System) is a hydraulic modeling software developed by the U.S. Army Corps of Engineers. It is designed to perform one-dimensional (1D) simulations of steady and unsteady flow, sediment transport, and water quality in riverine and open channel systems [9]. For 1D unsteady flow modeling, HEC-RAS solves the full dynamic Saint-Venant equations, which consist of the continuity and momentum equations [10], to simulate temporal and spatial variations in flow and water surface profiles along a river reach. In this approach, the flow is assumed to vary with time and along the channel length, while remaining laterally uniform within each cross-section. Required model inputs include river geometry, boundary conditions, initial conditions, and inflow hydrographs representing time-varying discharges.

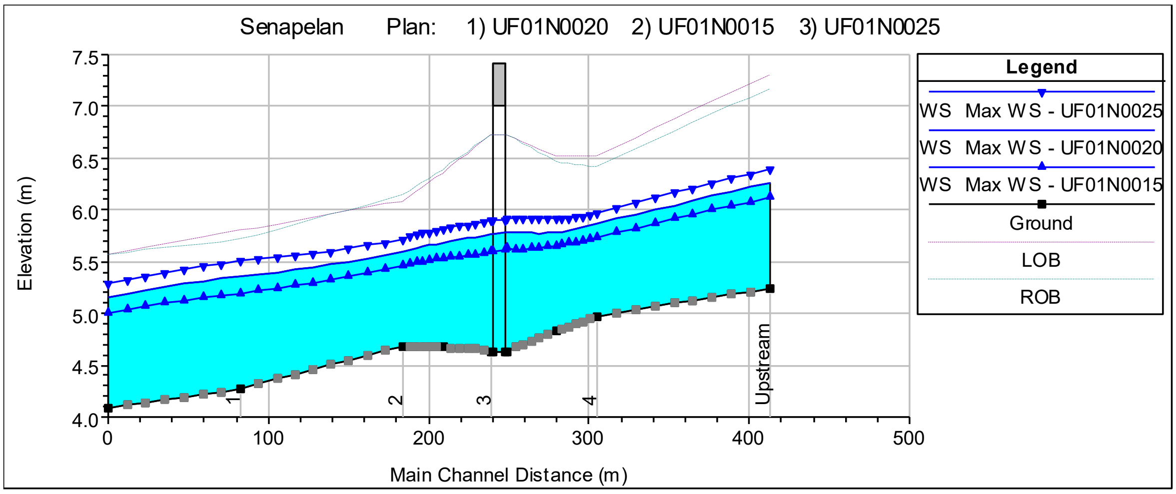


Source: Prepared by the authors.

Figure 2. Research Diagram

3. Results and Discussion

The initial assumed Manning's n value of 0.02 and the observed water level of 5.16 m were used to estimate the corresponding discharge. The downstream boundary condition was set using a normal depth with a slope of 0.00266 and a constant hydrograph variation. After a trial-and-error process, the resulting discharge was 10.8 m^3/s for a water level of 5.16 m. A sensitivity analysis of Manning's n was conducted using a range of plausible values: 0.015 and 0.025. The objective was to validate the assumed Manning's roughness coefficient. The resulting water levels from this analysis are presented in the figure below.



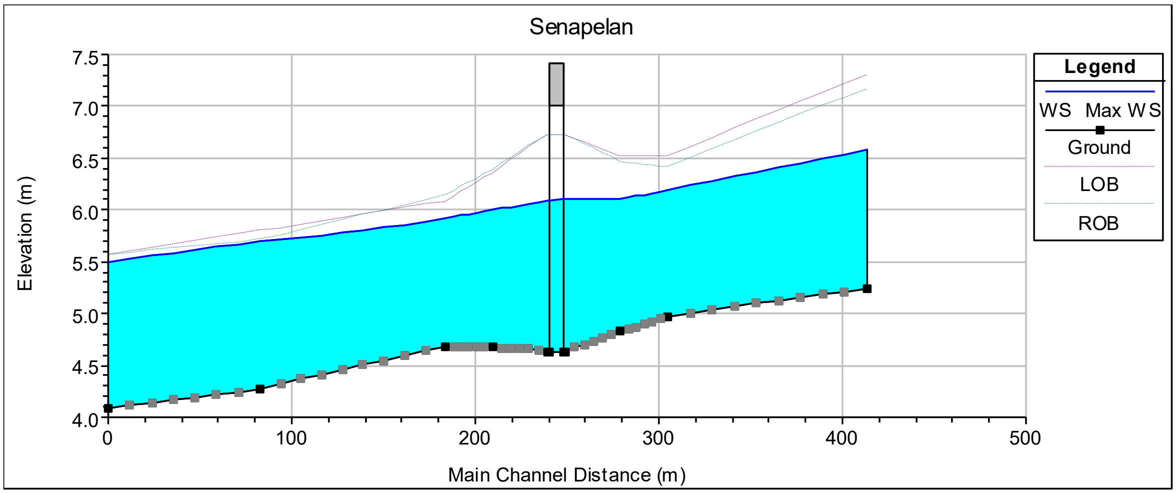
Source: Simulation results obtained in this study.

Figure 3. Water Surface Elevation from Manning's n Sensitivity Analysis

The results above demonstrated a clear inverse relationship: a smoother channel (n = 0.015) produced a lower water level (5.01 m), whereas a rougher channel (n = 0.025) produced a higher water level (5.29 m). Adjusting Manning's n coefficient by 0.005 resulted in a simulated water level change of approximately 13 cm. Based on field observations of the channel, which features side slopes constructed of plastered cement rubble masonry and a concrete bottom, an average composite Manning's n value of 0.020 was considered hydraulically representative of the existing physical conditions. This value is consistent with a channel exhibiting moderate roughness and minor surface irregularities, but not severe deterioration. It can be concluded that a discharge of approximately 10.8 m^3/s is the most probable flow rate associated with the observed water level during the measurement period, and the model is therefore considered validated under this specific flow condition.

3.1 Bankfull Flows

The resulting of bankfull capacity is presented in the figure below.



Source: Simulation results obtained in this study.

Figure 4. Bankfull Capacity Analysis

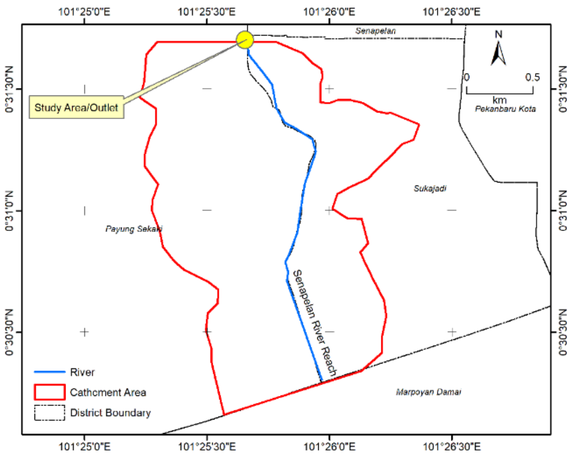
To evaluate the hydraulic thresholds of the river, a step-incremental unsteady flow simulation was conducted using ten discharge profiles ranging from 17.20 m^3/s to 19.00 m^3/s. The simulation results show that bankfull capacity varies spatially due to local channel geometry, with Cross-Section 4 acting as the primary bottleneck and Cross-Section 1 as a secondary vulnerability. At a baseline discharge of 17.20 m^3/s, the flow is contained within the main channel. However, increasing the discharge to 17.40 m^3/s causes overbank flooding at Cross-Section 4, raising the water level from 6.38 m to 6.59 m. This increase also breaches the lower right bank (5.71 m) at Cross-Section 1. In contrast, the downstream Cross-Section 0 shows higher capacity, safely containing flows up to 18.40 m^3/s before overtopping at 18.60 m^3/s, where the water level reaches 5.61 m. Since the total capacity of the river reach is limited by its narrowest or shallowest section, this study sets 17.20 m^3/s as the maximum safe bankfull capacity for this reach, as shown in figure 4.

3.2 Frequent and Extreme Flood Events

A hydrological analysis was conducted to determine return period of the design flood. The river reach watershed was derived from the downstream reach to the upstream reach of the river. The watershed delineation was produced using DEMNAS, along with the river reach's downstream threshold point. The results of this delineation are shown in Figure 5 below. The results of the watershed analysis are as follows: catchment area (A): 384 ha; river reach length (L): 2,858 m; and river reach slope (S): 0.004549. It should be noted that the slope value of

0.00266 used for the normal depth boundary condition represents the average bed slope of the local modeled river reach, derived from field-surveyed cross-section elevations. This differs from the slope (S) of 0.004549 reported in the watershed analysis, which represents the average slope of the full river length (L) of 2,858 m obtained from DEMNAS.

According to Regulation of the Minister of Public Works No. 12/PRT/M/2014, return period of design flood are determine by population and catchment area of the watershed. Based on Pekanbaru Municipality in Figures 2026, the population of Pekanbaru City are 1,192,990 people, and is classified as a metropolitan city. Therefore, the return period for flood discharge is between five and ten years. Ten years return period are selected as an extreme flood event. The results of the flood discharge calculations for various return periods are presented in Table 1. The resulting design flood discharges were plotted against their corresponding return periods to establish the flood magnitude–frequency relationship and to define the three representative hydraulic scenarios used in this study (Figure 6).



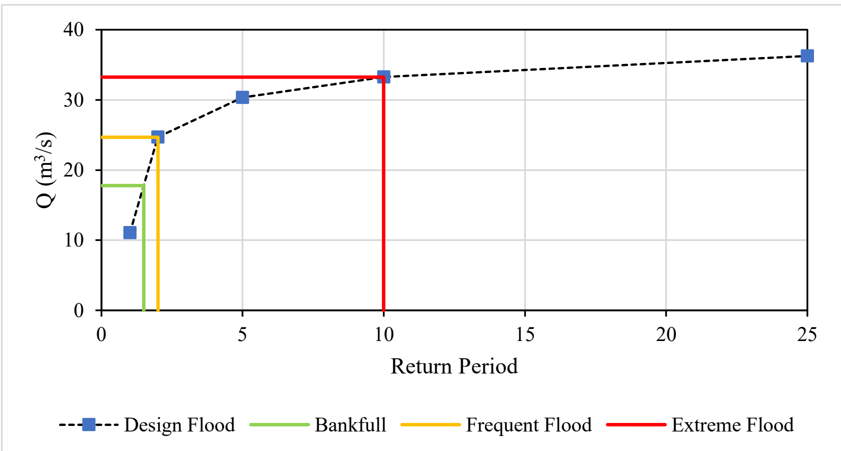
Source: Prepared by the authors.

Figure 5. Catchment Area Delineation

Table 1. Design Flood

Return Period	R (mm/day)	tc (hour)	I (mm/hour)	Q (m^3/s)
1.01	47.96	1.19	14.83	11.07
2	107.03	1.19	33.08	24.70
5	131.51	1.19	40.65	30.35
10	144.13	1.19	44.55	33.26
25	157.13	1.19	48.57	36.26

Source : Prepared by the authors.



Source: Prepared by the authors.

Figure 6. Design Flood Frequency Curve and Hydrological Capacity Benchmarks

Figure 6 shows that the estimated bankfull discharge (17.20 m^3/s) is lower than both the frequent flood discharge (24.70 m^3/s) and the extreme flood discharge (33.26 m^3/s). This indicates that the channel capacity is exceeded during both flood scenarios, suggesting that overbank flow is expected to occur once the discharge exceeds the bankfull condition. Based on the design flood frequency curve, the estimated bankfull discharge corresponds to a return period of approximately 1.5 years.

Because the field-surveyed cross-sections are strictly confined to the main channel banks, simulating discharges higher than the bankfull capacity (such as the 2-year and 10-year floods) within a 1D model would introduce artificial boundary errors, commonly known as the 'glass wall' effect. Furthermore, utilizing regional DEMNAS data to extend these profiles was rejected due to vertical datum discrepancies and the coarse spatial grid size relative to the narrow channel width. Therefore, rather than generating uncalibrated overbank maps, these higher discharges are utilized strictly as frequency benchmarks. As illustrated in Figure 6, the river's actual physical bankfull capacity (17.20 m^3/s) corresponds to a low return period of approximately 1.5 years. This key finding scientifically validates that the channel is severely under-capacity, as it fails to contain even a standard 2-year frequent flow event, providing clear evidence of the structural vulnerability causing the area's recurring urban flooding.

4. Conclusion

This study successfully characterised the hydraulic response of the Kedungsari section of the Senapelan River and determined the return period of its current capacity in conditions where data was scarce. 1D hydraulic modelling revealed that the physical bankfull capacity of the reach is strictly capped at 17.20 m^3/s, with Cross-Section 4 identified as the controlling hydraulic bottleneck. Integrating this capacity into the regional flood frequency curve shows that the existing channel corresponds to a return period of only approximately 1.5 years. This demonstrates that the existing channel is hydraulically undersized and highly susceptible to overbank flooding.

Under the Regulation of the Minister of Public Works No. 12/PRT/M/2014, urban drainage infrastructure is required to accommodate a minimum design flood of the 10-year return period. The Kedungsari reach of the Senapelan River, with a bankfull capacity of 17.20 m^3/s, is unable to convey the 2-year flood discharge of 24.70 m^3/s, much less the legally mandated 10-year discharge of 33.26 m^3/s. The channel therefore does not comply with the minimum hydraulic standard prescribed by this regulation. This non-compliance, combined with the reach's position in a densely populated urban area, warrants immediate structural intervention by the responsible water resources and municipal authorities.

This study demonstrates that integrating hydraulic bankfull capacity with design flood frequency benchmarks provides a practical framework for assessing flood thresholds in ungauged urban catchments where only main-channel geometry is available. Future research should use high-resolution UAV photogrammetry or LiDAR to safely extend cross sections beyond the main channel banks, avoiding vertical datum errors and enabling fully calibrated 2D flood inundation and hazard mapping.

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