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Capacity Analysis and Optimization of Urban Drainage Channels Using the SWMM: A Case Study of Jalan Raya Puputan

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Abstract: This study evaluates the performance of an existing urban drainage system and proposes a dimensional optimization strategy to improve its hydraulic capacity in the Plaza Renon area, Denpasar City. The analysis integrates hydrological and hydraulic approaches using 10 years of rainfall data (2015–2024). Hydrological analysis includes data quality assessment, rainfall frequency analysis using the Gumbel distribution, rainfall intensity estimation using the Mononobe method, and design discharge calculation using the Rational Method. Hydraulic performance is simulated using EPA SWMM 5.2. The developed model consists of 48 subcatchments, 67 junctions, 74 conduits, and 1 outfall, with acceptable simulation accuracy indicated by a flow continuity error of 0.01% and a surface runoff error of −0.11%. Under existing conditions, channel dimensions range from 0.30–0.63 m in width and 0.40–0.70 m in depth, resulting in capacity discharges of 0.14–1.03 m^3/s, which are insufficient compared to the design discharge (0.59–2.21 m^3/s). A redimensioning strategy is applied by increasing channel width to 1.2–1.5 m and depth to 1.4–1.5 m. The results show a substantial improvement in capacity discharge to 3.36–8.57 m^3/s, enabling all channels to safely convey the design discharge without overflow. This inadequacy leads to overflow in four critical channels. These findings demonstrate that channel redimensioning significantly enhances hydraulic performance and provides an effective solution for mitigating urban flooding in high-risk areas.

Keywords: Urban Drainage; SWMM 5.2; Drainage Channel Capacity; Flood Mitigation; Channel Redimensioning; Rainfall Intensity.

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1. Introduction

Rainfall is one of the essential components of the hydrological cycle and plays a significant role in human life, both in terms of resource availability and disaster potential [1]. High rainfall intensity can lead to increased surface runoff, which may result in inundation or flooding if not properly managed [2]. Rapid urbanization and increasing impervious surfaces significantly increase surface runoff and often exceed the capacity of existing urban drainage systems, resulting in frequent urban flooding events [3].Urban areas with high levels of urbanization are generally more vulnerable to flooding due to limited infiltration areas and reduced drainage capacity [4].

High-intensity rainfall events that have occurred in Denpasar City in recent years have caused inundation in several locations, including the Plaza Renon area. The existing drainage system has not been able to convey runoff optimally, resulting in water ponding on roadways and surrounding areas, particularly during prolonged rainfall events, thereby disrupting the activities of residents and visitors. Urban flooding frequently occurs when drainage infrastructure is unable to convey stormwater during extreme rainfall events [5].

The Plaza Renon area has a history of inundation triggered by high rainfall intensity and limited drainage capacity. Since 2020, flooding has occurred during heavy rainfall along Jalan Raya Puputan, with more significant events recorded on 27 December 2025 [7] and again on 09 January 2026 [8]. These recurring incidents indicate that Plaza Renon is a flood-prone area and requires a comprehensive evaluation of the existing drainage system performance.

Recurring inundation indicates that the drainage channel capacity is insufficient to accommodate runoff discharge optimally, thereby necessitating an evaluation of the existing system's condition and performance [8]. This study employs the EPA SWMM to simulate runoff and flow within the drainage network based on hydrological and hydraulic parameters [9]. The Storm Water Management Model (SWMM) is widely used to simulate rainfall–runoff processes and evaluate the performance of urban drainage systems under various hydrological conditions [10]. Through this modeling approach, a quantitative assessment of drainage performance is obtained, serving as a basis for identifying flood-prone locations and formulating more targeted, data-driven improvement recommendations. This study aims to evaluate the capacity and performance of the existing drainage system in the Plaza Renon area using the SWMM, and to propose optimization measures to reduce inundation risk along Jalan Raya Puputan.

2. Methods

This study was conducted from October 2025 to February 2026. The research location is situated along Jalan Raya Puputan, specifically in the Plaza Renon area, at approximately 8°40′24″S and 115°14′39″E. The map of the study area is presented in **Figure 1**.

Figure 1. Study Area Location Map

2.1. Materials

The materials used in this study include data on the dimensions and cross-sectional types of drainage channels, as well as monthly rainfall data over a 10-year period obtained from the Meteorology, Climatology, and Geophysics Agency (BMKG). These data served as the basis for hydrological analysis and drainage capacity evaluation. The software utilized in this research includes Google Earth for measuring subcatchment areas, channel lengths, and preliminary channel elevations; ArcGIS (licensed software) for land cover analysis; EPA SWMM 5.2 for hydrological and hydraulic simulations; and

Microsoft Excel for data processing. Channel elevation data obtained from Google Earth are considered secondary data and were used as an initial approximation of the drainage profile, with recognized limitations in vertical accuracy.

2.2. Research Procedures

This research was conducted through several systematic and interconnected stages to achieve the objectives of the study. The research stages began with problem formulation, literature review, and data collection, followed by data processing and analysis consisting of hydrological and hydraulic analyses. These stages were structured to describe the overall research procedure carried out to evaluate and optimize the performance of the drainage system in the study area.The research flowchart is illustrated in Figure 2.

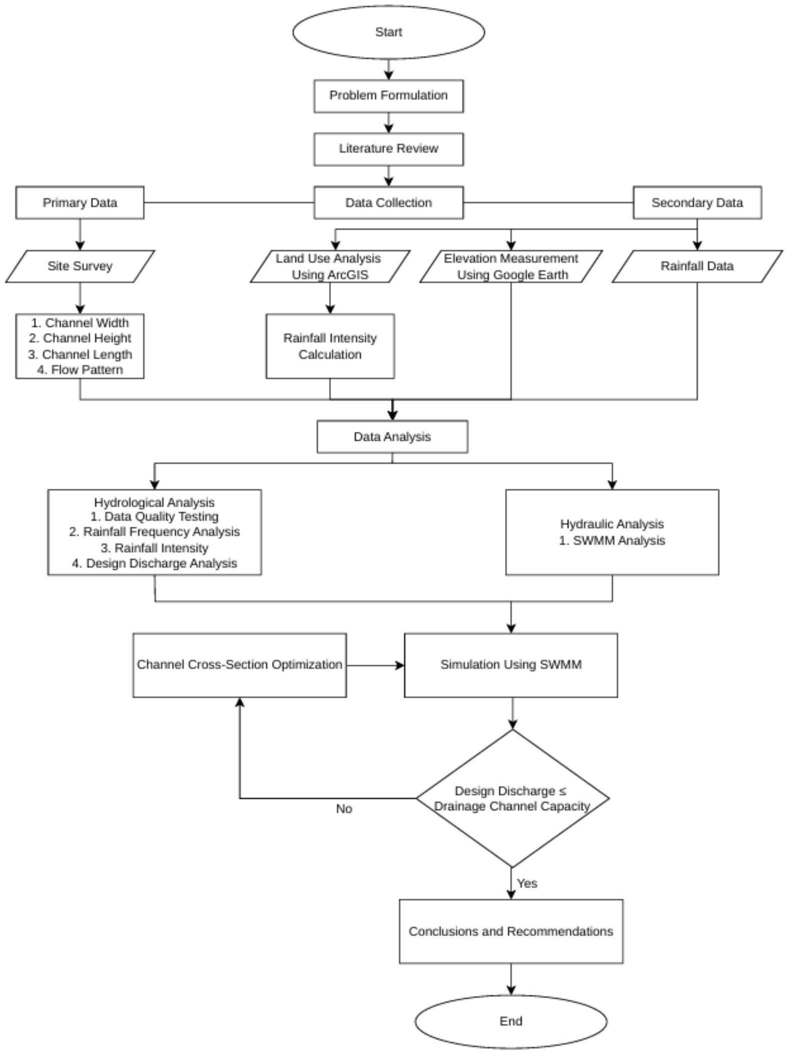


Figure 2. Research Procedure Flowchart

2.2.1 Survey

The survey was conducted in stages, beginning with tracing the flow patterns of the drainage channels, followed by measuring the existing channel dimensions. The results indicate that several channel segments experience sedimentation ranging from mild to severe, consisting of sand deposits, dense vegetation (grass), and plastic waste. These conditions reduce the channel capacity to convey runoff, thereby contributing to the occurrence of inundation.

2.2.2 Data Collection

The data collection stage in this study comprised both primary and secondary data obtained in accordance with the analysis requirements. Primary data were collected through direct field surveys, including measurements of channel dimensions (width, height, and cross-sectional shape), channel length, channel invert elevation, as well as identification of flow direction and flow patterns to represent the existing condition of the drainage system. Secondary data consisted of rainfall records from official observation stations and land use data analyzed using ArcGIS, as well as channel elevation measurements obtained through Google Earth.

2.2.3 Data Processing and Analysis

The data processing and analysis stage was carried out through two main phases, namely hydrological analysis and hydraulic analysis. The hydrological analysis included rainfall data quality testing, areal rainfall analysis, rainfall frequency analysis, rainfall intensity analysis, and design discharge estimation using the Rational Method. Subsequently, hydraulic analysis was performed through modeling and simulation of the drainage system using SWMM to compare the design discharge with the channel capacity. SWMM enables detailed simulation of runoff generation, flow routing, and hydraulic performance of drainage networks through components such as subcatchments, junctions, conduits, and outfalls. If the design discharge was less than or equal to the channel capacity, the study proceeded to the conclusion stage; otherwise, if it exceeded the capacity, channel cross-sections were optimized and re-simulated using SWMM until satisfactory results were obtained.

3. Result and Discussion

3.1 Rainfall Data Quality Testing

The rainfall data used in this study consisted of 10 years (2015–2024) of secondary data obtained from the Sanglah Station. Prior to analysis, rainfall data quality testing was conducted to ensure data reliability and suitability. The tests included consistency testing (RAPS), trend absence testing, stationarity testing, persistence testing, and outlier testing. The results of the rainfall data quality assessment are presented in **Table 1**.

Table 1. Recapitulation of Rainfall Data Quality Testing Results at Sanglah Station

Sanglah Station				
Consistency Test (RAPS)	Trend Absence Test	Stationarity Test	Persistence Test	Outlier Test
The data are consistent.	No trend was detected.	The variance value is stable.	The data show no dependency.	No outliers were identified in the data.

Table 1 presents a recapitulation of the rainfall data quality assessment at Sanglah Station, including the consistency test (RAPS), trend absence test, stationarity test, persistence test, and outlier test. The RAPS results indicate that the data are consistent, suggesting the absence of systematic deviations throughout the observation period. The trend absence test using the Spearman method, a non-parametric statistical analysis designed to determine whether a dataset exhibits a trend, shows that no significant trend was detected in the data presented in **Table 1** [11]. The stationarity test demonstrates stable variance, indicating that the statistical parameters of the data remain relatively constant over time. The persistence test reveals no significant serial dependence, confirming that the data satisfy the assumption of independence. In addition, the outlier test was conducted to identify potential data deviations based on upper and lower threshold limits; values exceeding these limits are classified as

outliers due to their significant deviation from the main distribution pattern [12]. Overall, the rainfall data are considered suitable for subsequent hydrological analysis.

3.2 Maximum Monthly Rainfall Analysis

The analysis was conducted using the arithmetic mean approach to identify the distribution of maximum monthly rainfall based on the time period, thereby indicating the potential occurrence of the highest rainfall [13]. The results show that the highest average rainfall occurred in February at 85.5 mm, while the lowest was recorded in August at 12.93 mm.

Table 2. Maximum Monthly Rainfall

Year	JAN	FEB	MAR	APR	MEI	JUN	JUL	AGT	SEP	OKT	NOP	DES
2015	56.5	74.2	98.6	9.4	38.4	0.3	0.8	5.8	0.4	0	6	83.2
2016	32.7	180	46.5	30.5	12.5	47.5	31.2	12.8	76.8	38.4	65.2	70.5
2017	62.5	106	25	43.2	32.2	38.4	9.4	14	3.3	33.5	92.5	86.5
2018	138.7	35.5	51.5	0.8	3.7	19.5	16.2	12.7	6.5	10.5	133	39.4
2019	72.5	30.4	69.5	24.2	2.6	0.9	1.3	8.8	9.5	0	17.8	48.9
2020	72.6	123.4	48.5	25.4	19.8	79	14	1.7	23.2	60.4	13.1	83.7
2021	143.3	77.7	48	27.4	1.2	82.5	5.9	47.1	113	50.1	74	183.5
2022	57	80.5	91	64	32	19.3	8.6	18	49	120	55.5	55
2023	58.8	61.5	16	30.5	17	37.9	94.2	5.6	1.6	0	51.2	90.2
2024	46.9	85.8	95.4	60.8	1.6	15.5	26.5	2.8	36	74.2	41.8	101.4
Sum	741.5	855	590	316.2	161	340.8	208.1	129.3	319.3	387.1	550.1	842.3
Average	74.15	85.5	59	31.62	16.1	34.08	20.81	12.93	31.93	38.71	55.01	84.23

Table 2 presents the maximum monthly rainfall data for the 2015–2024 period, which serve as the basis for analyzing rainfall characteristics in the study area. The values in the “Total” row represent the cumulative maximum monthly rainfall over the ten-year observation period, while the “Average” row was calculated using the arithmetic mean method to describe the tendency of maximum rainfall intensity.

3.3 Rainfall Frequency Analysis

The maximum monthly rainfall data obtained over the 10-year observation period were subsequently analyzed to determine the design rainfall using a probability distribution approach. Based on the evaluation results, the Gumbel distribution was found to satisfy the required criteria, with a skewness coefficient (Cs) ≤ 1.1306 and a kurtosis coefficient (Ck) ≤ 5.40 in the frequency analysis [14]. The Gumbel method was further tested for goodness-of-fit using the Smirnov–Kolmogorov test and the Chi-Square test. The objective of frequency analysis is to estimate the magnitude of an event and its frequency or return period using probability distributions [15]. The results of the distribution testing are presented in **Table 3**.

Table 3. Determination of Rainfall Distribution

No	Type of Distribution	Criteria	Calculation Results	Conclusion
1	Gumbel	Cs ≤ 1.1306	0.78	Meets the criteria
		Ck ≤ 5.40	3.76	Meets the criteria
2	Log Normal	$Cs = Cv^3+3Cv$	0.92	Does not meet the criteria
		$Ck = Cv^8+6Cv^6+15Cv^4+16Cv^2+3$	4.55	Does not meet the criteria

No	Type of Distribution	Criteria	Calculation Results	Conclusion
3	Normal	Cs = 0	0.78	Does not meet the criteria
		Ck = 3	3.76	Does not meet the criteria
4	Log Pearson Tipe III	In addition to the values above	-	Meets the criteria

Table 3 presents the results of the probability distribution suitability test based on the skewness coefficient (Cs) and kurtosis coefficient (Ck) parameters. The Gumbel distribution meets the criteria, as the Cs value of 0.78 and the Ck value of 3.76 fall within the theoretical limits. The Log-Normal and Normal distributions do not satisfy the required parameter criteria. Meanwhile, the Log-Pearson Type III distribution is considered acceptable due to its flexibility with respect to the Cs and Ck values.

Based on the chi-square test table, the critical value (χ^2cr) is 5.991, which is greater than the calculated χ^2 value of 1 obtained from **Table 2**. These results indicate that the Gumbel distribution passes the goodness-of-fit test and can be accepted. Furthermore, the Gumbel method was evaluated using the Smirnov–Kolmogorov test, and the results show that the critical D value (Do) is greater than the maximum D value (D). The calculated maximum D value is 0.10334, while the critical D value is 0.409. Therefore, the Gumbel method is considered acceptable.

Table 4 Chi-Square Test Results for the Gumbel Distribution

Class	Interval					Oi	Ei	(Ei-Oi)	$(Ei-Oi)^2$	$(Ei-Oi)^2/Ei$
1			P	<	96.61	2	2	0	0	0
2	96.61	<	P	<	111.77	3	2	1	1	0.5
3	111.77	<	P	<	127.50	2	2	0	0	0.00
4	127.50	<	P	<	149.79	1	2	-1	1	0.50
5			P	>	149.79	2	2	0	0	0.00
Total						10	10	0	2	1

Table 4 presents the results of the Chi-Square test for the Gumbel distribution by comparing the observed frequency (Oi) and the expected frequency (Ei) in each interval class [16]. The total value of $(Ei - Oi)^2/Ei$ is 1, which represents the calculated χ^2 value (χ^2cal). Since the χ^2cal value is smaller than the critical χ^2 value (χ^2cr), the Gumbel distribution is considered acceptable and consistent with the maximum rainfall data [17].

Table 5 Maximum Rainfall Using the Gumbel Method

Return Period Year	Yt	K	Rmax (mm)
2	0.3665	-0.135531	122.69733
5	1.4999	1.058024	124.33393
10	2.2502	1.848147	126.9057
20	2.9606	2.596251	130.43346
25	3.1985	2.846778	131.85242
50	3.9019	3.587511	136.74511
100	4.6001	4.322767	142.63216

Table 5 presents the results of the design maximum rainfall calculation using the Gumbel method for various return periods. The Rmax values were obtained from the mean data adjusted by the frequency factor (K) based on the reduced variate (Yt). It can be observed that as the return period increases, the Rmax value also increases. However, the Rmax values were not directly used as input in the SWMM simulation. Instead, these values were used as the basis for calculating rainfall intensity using the Mononobe method, which was then applied as the design rainfall input in the SWMM model.

3.4 Rainfall Intensity Analysis

Rainfall intensity analysis refers to the depth of rainfall occurring over a specific period during which the precipitation is concentrated [18]. The relationship between rainfall intensity and rainfall duration can be determined using several empirical approaches, such as the Mononobe formula. Based on previous studies, rainfall events in Indonesia typically have an average duration of approximately six hours, although this may vary depending on regional climatic conditions [19].

Table 6 Rainfall Intensity Calculation Results Using the Mononobe Method

t(hour)	Return Period					
	2	5	10	25	50	100
1	42.537	43.104	43.996	45.711	47.407	49.448
2	26.796	27.154	27.716	28.796	29.864	31.150
3	20.450	20.722	21.151	21.975	22.791	23.772
4	16.881	17.106	17.460	18.140	18.813	19.623
5	14.547	14.741	15.046	15.633	16.213	16.911
6	12.882	13.054	13.324	13.844	14.357	14.975

Based on **Table 6**, the rainfall intensity results obtained using the Mononobe method indicate that as the return period increases, the rainfall intensity tends to increase for each rainfall duration [20]. Conversely, as the rainfall duration (t) becomes longer, the rainfall intensity decreases for all return periods. This pattern demonstrates that rainfall events with longer return periods have the potential for higher intensities,particularly over short durations, thereby significantly influencing the estimation of design discharge and the dimensioning of drainage channels [16].

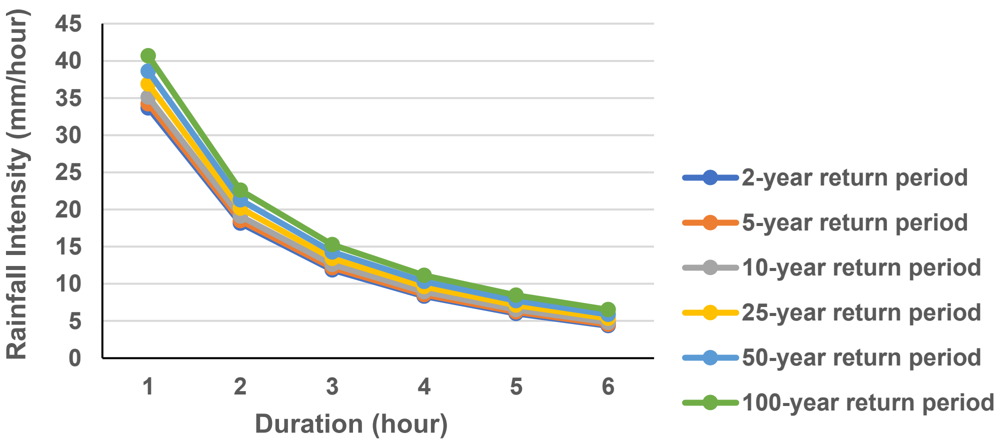


Figure 3. IDF Curve (Intensity–Duration–Frequency Curve)

Based on **Figure 3**, the rainfall intensity curve shows that rainfall intensity decreases as rainfall duration increases for all return periods. Conversely, for the same duration, a longer return period corresponds to a higher rainfall intensity. This indicates that rainfall events with longer return periods exhibit more extreme characteristics, particularly over short durations, making them a critical factor in determining design discharge and evaluating drainage channel capacity [21].

3.5 Drainage Channel Evaluation Using the SWMM 5.2 Program

The drainage network modeling in the Plaza Renon area was conducted using the SWMM version 5.2 software. Before conducting the modeling, the study area was divided into subcatchments based on drainage flow patterns and connectivity within the system. Each subcatchment represents an area that contributes runoff to a specific junction, determined by surface slope, flow direction, and the configuration of the existing drainage network. In densely developed areas, smaller subcatchments were defined to represent detailed flow distribution, while larger subcatchments were applied in less complex areas. The developed model represents several main components of the drainage system, namely the catchment areas (subcatchments), flow nodes (junctions), the final discharge point (outfall), and the conveyance channels (conduits) [14]. Based on the modeling results, the drainage system in the study area consists of 48 subcatchment units (S), 67 junctions (J), 1 outfall (OUT), and 74 conduits (C). During the simulation process, each subcatchment was entered into the model with several input parameters representing the hydrological characteristics of the catchment area [22].

Subsequently, the drainage network was modeled using the SWMM 5.2 software, as shown in **Figure 4**. The simulation results are considered acceptable because the continuity error value for flow routing is 0.01%, while the surface runoff continuity error is −0.11%. These values are still within the acceptable tolerance range, considering that calibration results are generally regarded as good when the error level is less than 10%.

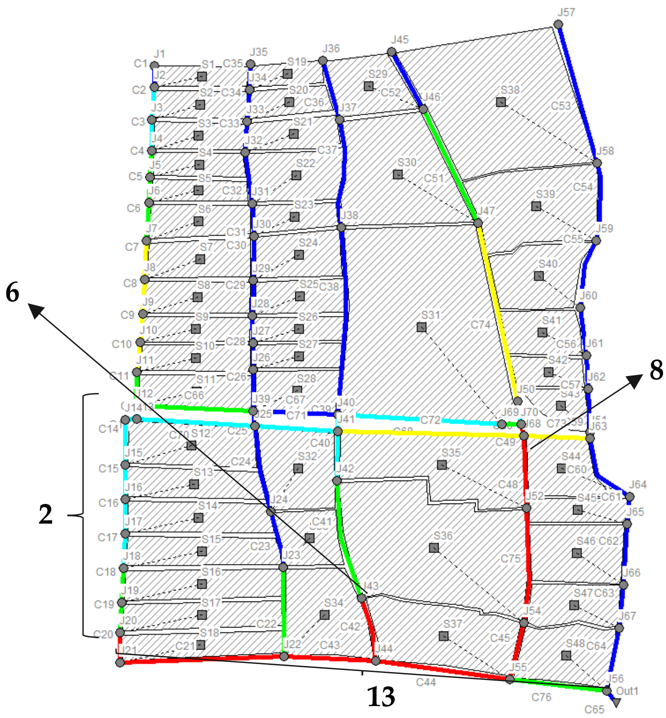


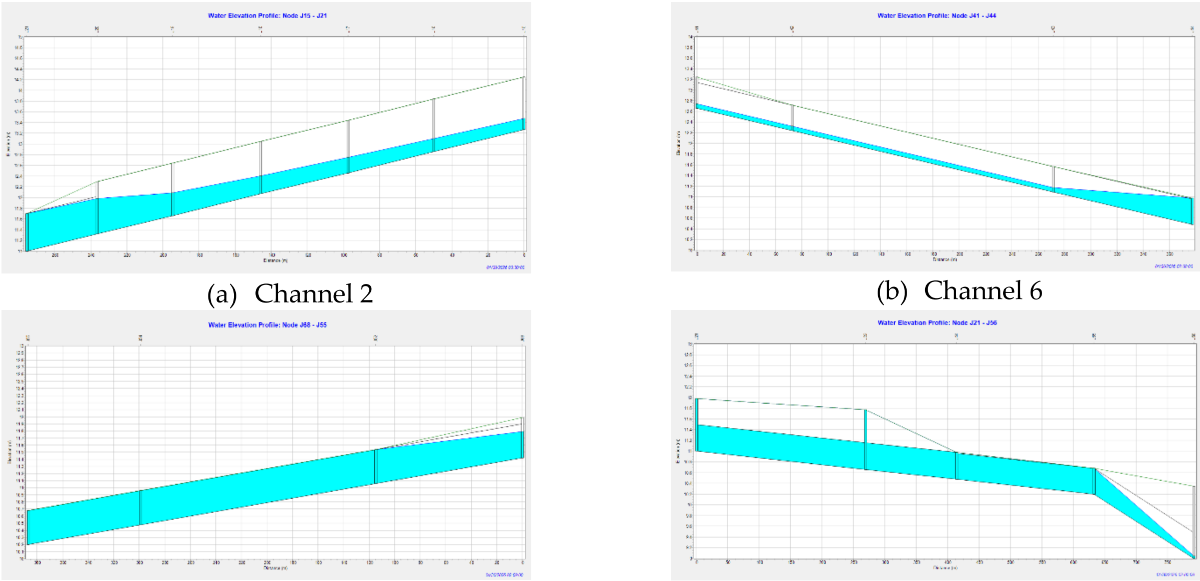
Figure 4. Drainage System Modeling

Table 7. Existing Condition

Channel	Existing Condition
2	
6	
8	
13	

Based on the simulation results using a 10-year return period rainfall, after improving the drainage channel elevations, there are still four channels experiencing overflow (describe how its overflow?), namely Channel 2 (C20), Channel 6 (C42), Channel 8 (C48, C75, C45), and Channel 13 (C21, C43, C44). The overflow occurred when the simulated runoff discharge exceeded the available channel capacity, causing the water level to rise above the channel walls and spill onto the surrounding road surface. This condition mainly occurred during peak rainfall events due to flow accumulation in the downstream sections and limited channel dimensions.This indicates that the channel capacities are insufficient to accommodate the flow discharge. Therefore, the channels are likely to overflow during peak rainfall periods. The high runoff volume and low infiltration rate in each subcatchment are factors contributing to the occurrence of runoff, causing most of the rainfall to flow as surface runoff rather than being absorbed into the soil [23]. The infiltration value used in the subcatchments is 2.34 mm.

The overflow in Channel 2 occurs because its dimensions are insufficient, causing the runoff to exceed the channel capacity. Channel 2 has a width of 0.58 m and a depth of 0.70 m. Node J15 serves as the inlet node and J21 as the outlet node of Channel 2. Figure 5 shows that the flow profile has reached its maximum capacity and is unable to accommodate the high rainfall discharge, which may result in runoff and potential flooding.



(a) Channel 2 (b) Channel 6

(c) Channel 8 (d) Channel 13

Figure 5. Channel Flow Profile Condition

3.6 Optimization of Drainage Channels Using the SWMM 5.2 Program

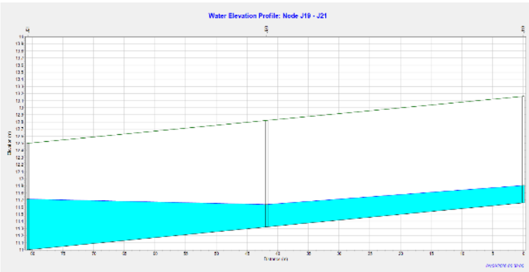
Improvements were carried out on four channels, namely Channel 2 (C20), Channel 6 (C42), Channel 8 (C48, C75, C45), and Channel 13 (C21, C43, C44). The improvement was conducted by redimensioning the channels through increasing the channel width and depth in order to enhance the flow capacity (24). **Table 8** presents the comparison of channel dimensions between the existing condition and the improved condition.

Table 8 Results of the Drainage Channel Dimension Design

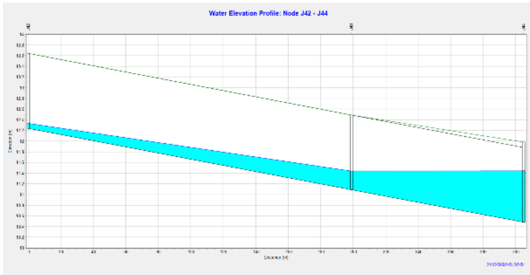
Conduit	Existing Condition				Improved Dimension Condition			
	Dimensions (m)		Design Discharge (m3/s)	Capacity Discharge (m3/s)	Dimensions (m)		Design Discharge (m3/s)	Capacity Discharge (m3/s)
	Width	Depth			Width	Depth		
C20	0.58	0.70	1.35	0.75	1.5	1.5	1.35	7.56
C21	0.58	0.50	1.37	0.57	1.5	1.5	1.37	8.57
C42	0.30	0.40	1.30	0.22	1.2	1.4	1.30	4.78
C43	0.58	0.50	0.59	0.21	1.5	1.5	0.59	3.36
C44	0.58	0.48	1.22	0.52	1.5	1.5	1.22	3.36
C45	0.58	0.48	1.22	0.52	1.2	1.4	1.22	5.53
C48	0.58	0.48	1.29	0.39	1.2	1.4	1.29	5.53
C75	0.58	0.48	1.96	0.52	1.2	1.4	1.96	5.53

The table shows that under existing conditions, the channel dimensions are relatively small, with widths ranging from 0.30 to 0.63 m and depths from 0.40 to 0.70 m. The design discharge ranges from 0.59 to 2.21 m³/s, while the channel capacity discharge only ranges from 0.14 to 1.03 m³/s, indicating that most channels are unable to accommodate the design discharge. After dimensional improvements, the channel width increases to 1.2–1.5 m and the depth to 1.4–1.5 m. The improved dimensions were determined through an iterative trial-and-error process in the SWMM model by gradually adjusting channel width and depth until all channels were able to safely convey the design discharge without overflow. The final dimensions were selected based on the minimum effective size required to achieve optimal hydraulic performance and were adjusted to commercially available standard U-Ditch dimensions to ensure practical implementation in the field.The capacity discharge shows a significant increase, ranging from 3.36 to 8.57 m³/s, such that all channels are able to exceed the design discharge, which remains within the same range. This demonstrates that the dimensional improvements effectively enhance flow capacity and the overall performance of the drainage system.

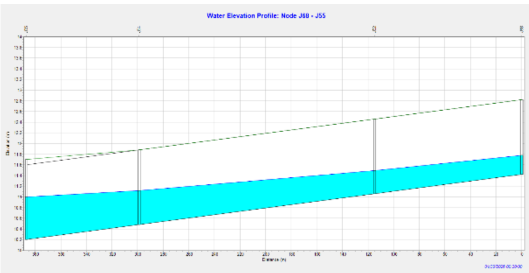
Before the redimensioning process, the channel invert elevations were adjusted to ensure a consistent longitudinal profile and proper flow direction. The subsequent redimensioning was carried out by increasing channel width and depth while maintaining the improved longitudinal profile, thereby preserving a stable hydraulic gradient and preventing flow disturbances. The channel flow profile after redimensioning is shown in Figure 6. The simulation results indicate that after the modification of the channel dimensions, the water flow can be adequately conveyed within the channel cross-section. The water level remains below the channel boundary, allowing the flow to pass through the channel without causing overflow [25].



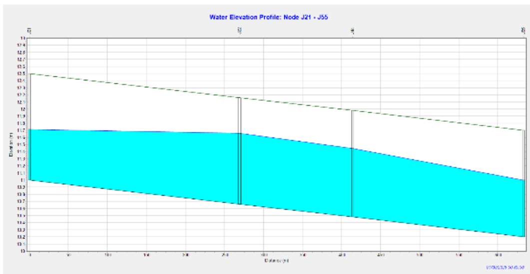
(a) Channel 2



(b) Channel 6



Channel 8



Channel 13

Figure 6 Channel Flow Profile After Channel Redimensioning

4. Conclusion

Based on the results of hydrological analysis and hydraulic simulation using the EPA SWMM 5.2 program, the following conclusions can be drawn:

1. The existing drainage system in the Plaza Renon area is hydraulically inadequate to convey peak runoff. The channel capacity ranges from 0.14 to 1.03 m³/s, which is significantly lower than the design discharge of 0.59 to 2.21 m³/s. This condition results in overflow in several critical channels, namely Channel 2 (C20), Channel 6 (C42), Channel 8 (C48, C75, C45), and Channel 13 (C21, C43, C44).
2. The implementation of channel redimensioning, by increasing the width to 1.2–1.5 m and the depth to 1.4–1.5 m, significantly enhances the hydraulic performance of the system. The channel capacity increases to 3.36–8.57 m³/s. This improvement allows all channels to accommodate the design discharge and eliminates overflow during peak rainfall conditions.
3. The results demonstrate that channel dimension is a critical factor influencing drainage performance. The redimensioning strategy not only increases flow capacity quantitatively but also provides a sufficient safety margin against the design discharge. Therefore, channel redimensioning is recommended as an effective and practical solution for improving urban drainage systems and reducing flood risk in flood-prone areas.

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