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Evaluation of Jongaya Drainage System in Makassar Due to Channel Network Changes

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ABSTRACT

Makassar faces flooding challenges due to rapid urban development that lack of effective planning. Makassar faces heightened flood risks due to inadequate drainage infrastructure, particularly in areas with poor drainage systems and elevations between 0 and 4 meters above sea level. The city experiences substantial rainfall annually, as happened on February 13, 2023, which causes flooding at several points and is made worse by high tides. The study aims to investigate the existing conditions of the drainage channel in the Jongaya drainage system as well as evaluate the alternative condition proposed by the Public Works Agency of Makassar, assess planned flood discharge, determine channel dan Jeneberang Lama long storage capacity, and identify necessary drainage system support facilities. Based on the simulation analysis results using Storm Water Management Model (SWMM) 5.2, flooding still occurs in the existing condition with a volume of 169,521 m³, as well as in the alternative condition with a volume of 148,477 m³. An evaluation was conducted for both conditions involving redesigning cross-sections, adjusting elevations, and planning 4 pumps and 2 sluice gates at the storage area outlet, which subsequently eliminated flooding in both conditions. The flood design observed in both conditions are 103.43 m³/s and 77.48 m³/s respectively.

1. Introduction

Makassar, the provincial capital of South Sulawesi in Indonesia, is a coastal city situated between the Makassar Strait and the Jeneberang River [1]. Covering approximately 175.77 square kilometers and inhabited by around 1,432,189 residents [2], it is one of eastern Indonesia's largest cities. Despite its status as a major economic hub, Makassar faces recurring and significant flooding challenges. Population growth, driven by a 0.97% increase by 2023 due to high urbanization rates, has intensified demand for housing and infrastructure, leading to the conversion of natural absorbent surfaces into impermeable ones [3]. This urban development without effective planning has increased surface water flow and reduced land

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absorption capacity [4][5]. In 2022, green open space (RTH) in Makassar constituted only 9.07% of the city's total area, far below the 30% mandated by Undang-Undang No. 26 of 2007 [6]. Additional factors contributing to flooding include poor drainage channels and low land elevation [7], [8].

Makassar's primary drainage system relies on the Pannampu, Jongaya, and Sinrijala channels, serving the city from south to north [9]. The Jongaya channel, crucial for downtown Makassar, frequently floods during the rainy season, particularly during events like the severe flood on February 13, 2023, the worst in 20 years [10]. Contributing factors include sedimentation, debris accumulation, inadequate secondary channel capacity, and poor interconnections between channels. A 2014 Feasibility Study by the Dinas Pekerjaan Umum Kota Makassar identified several flood-prone areas and prioritized them into seven subsystems, including the Jongaya canal drainage system [11]. Although critical inundation control measures were planned for the Jongaya system, implementation has yet to occur, and the area remains flood-prone as of 2020.

Addressing Makassar's flooding issues requires evaluating the existing drainage system to determine if changes in channel dimensions, system modifications, or additional drainage support facilities are needed. This research focuses on investigating the existing Jongaya drainage system and assessing the planned changes proposed by the Public Works Agency of Makassar City in 2014 which in this research is referred to as alternative conditions, determine channel cross-sectional capacities and volume capacity of the long storage in Jeneberang Lama resulting from changes in the drainage system, identify necessary drainage system support facilities [11]. The goal is to prevent or reduce flooding and waterlogging risks during the rainy season.

2. Research Method

Evaluation of the Jongaya drainage system began with a field study and literature review stage. Field studies and resident interviews were conducted to inspect the Jongaya drainage system and address annual flooding issues. These studies, combined with literature reviews on hydrology, hydraulics, and SWMM 5.2 simulations, provide a theoretical framework for this research. The SWMM assesses the drainage design using discharge data derived from the Rational Method. This model is reliable in generating the runoff assigned to each sub-catchment parameter that receives the rainfall, and it is possible to simulate the dynamic rainfall runoff in urban areas from single and continuous peak runoff [12].

This research utilizes secondary data including hydrological data (rainfall), spatial data (DEMNAS, drainage layout, land use, rain station), and hydraulic data (channel cross-sections, elevations, storage volume). Data processing involves hydrological and hydraulic analysis. Hydrological analysis includes average rainfall analysis using the Polygon Thiessen method, frequency distribution analysis, goodness-of-fit tests, calculation of planned rainfall for return periods, and hourly rainfall distribution input into the Storm Water Management Model (SWMM). The hydraulic analysis uses the EPA SWMM 5.2 program. The process includes project setup, inserting a backdrop map, drawing objects (subcatchments, junctions, conduits, outfalls), editing object properties, running simulations, and evaluating results. This identifies channels experiencing flooding and assesses the volume capacity of storage units to determine necessary supporting facilities.

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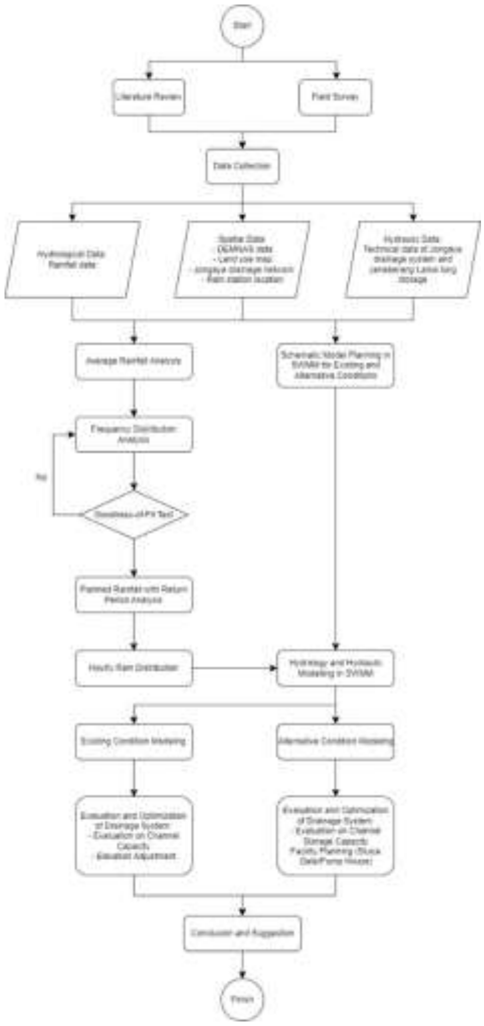

Source: Personal Documentation (2024)

Figure 1. Research Flow Chart

3. Results and Discussions

3.1 Average Rainfall Analysis

Average rainfall analysis utilizes the Thiessen polygon method, examining three rain stations: Paotere, Geofisika Gowa, and Nipa-Nipa. Analysis results indicate that Paotere and Gowa Geophysics are the most influential stations. The study uses average maximum rainfall data, derived from daily rainfall records from 2014 to 2023.

Table 1. Maximum Rainfall for Existing Condition

Year	Stations		Average	Max
	Paotere Coeff. = 0.63	Geof.Gowa Coeff. = 0.37		
2014	29.1	77	46.60	46.60
	0	62	22.65	
2015	88	206	131.11	131.11
	160.3	0	101.73	
2016	160.3	137	151.79	151.79
	188.7	44	135.83	
2017	109.3	86	100.79	128.96
	156	82	128.96	

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Year	Stations		Average	Max
	Paotere	Geof.Gowa		
	Coeff. = 0.63	Coeff. = 0.37		
2018	52.2	159	91.22	141.10
	152.1	122	141.10	
2019	86.2	125	100.38	100.38
	107	0	67.91	
2020	122.9	0	78.00	78.00
	0.1	139	50.85	
2021	145.9	187	160.92	160.92
	218.8	14.5	144.15	
2022	20	154	68.96	136.37
	155.3	103.5	136.37	
2023	42.1	158	84.45	172.23
	200	124	172.23	

Source: Analysis Result (2024)

Table 2. Maximum Rainfall for Alternative Condition

Year	Stations		Average	Max
	Paotere	Geof.Gowa		
	Coeff. = 0.49	Coeff. = 0.51		
2014	29.1	77	53.44	53.44
	0	62	31.51	
2015	88	206	147.97	147.97
	160.3	0	78.83	
2016	160.3	137	148.46	148.46
	188.7	44	115.16	
2017	109.3	86	97.46	118.39
	156	82	118.39	
2018	52.2	159	106.48	136.80
	152.1	122	136.80	
2019	86.2	125	105.92	105.92
	107	0	52.62	
2020	122.9	0	60.44	70.69
	0.1	139	70.69	
2021	145.9	187	166.79	166.79
	218.8	14.5	114.97	
2022	20	154	88.10	128.97
	155.3	103.5	128.97	
2023	42.1	158	101.00	161.37
	200	124	161.37	

Source: Analysis Result (2024)

3.2 Frequency Distribution Analysis

Frequency distribution analysis is used to obtain the rainfall design using rainfall data from the past 10 years [13]. This analysis includes calculating statistical parameters using the Gumbel, Normal, Log Normal, and Log Pearson Type III distribution methods, which are then compared with the parameter requirements of each distribution method to determine which method is acceptable to apply [14].

Based on the calculation results in Table 3 and Table 4, it can be concluded that Log Pearson Type III Distribution Method fulfills the requirements in both conditions.

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Table 3. Recapitulation of Statistical Parameter Requirements for Existing Condition

Distribution Method	Requirement	Result		Conclusion
Gumbel	Cs = 1.14	Cs	-0.49	NOT OK
	Ck = 5.402	Ck	3.84	NOT OK
Normal	Cs = 0	Cs	-0.49	NOT OK
	Ck = 3	Ck	3.84	NOT OK
Log Normal	Cs = 3 or Cs=3Cv	Cs	-0.98	NOT OK
		Ck	4.66	NOT OK
Log Pearson Type III	Cs and Ck flexible	Cs	-0.98	OK
		Ck	4.66	OK

Source: Analysis Result (2024)

Table 4. Recapitulation of Statistical Parameter Requirements for Alternative Condition

Distribution Method	Requirement	Result		Conclusion
Gumbel	Cs = 1.14	Cs	-0.49	NOT OK
	Ck = 5.402	Ck	3.84	NOT OK
Normal	Cs = 0	Cs	-0.49	NOT OK
	Ck = 3	Ck	3.84	NOT OK
Log Normal	Cs = 3 or Cs=3Cv	Cs	-0.98	NOT OK
		Ck	4.66	NOT OK
Log Pearson Type III	Cs and Ck flexible	Cs	-0.98	OK
		Ck	4.66	OK

Source: Analysis Result (2024)

3.3 Goodness-of-Fit Test

Average rainfall analysis utilizes the Thiessen polygon method, examining three rain stations: Paotere, Geofisika Gowa, and Nipa-Nipa. Analysis results for both existing and alternative conditions indicate that Paotere and Gowa Geophysics are the most influential stations. The study uses average maximum rainfall data, derived from daily rainfall records from 2014 to 2023. The results of the test showed that the design rainfall by the calculation of the Pearson Type III frequency distribution was acceptable.

3.4 Design Rainfall with Return Period

Based on the goodness-of-fit test results, the Log Pearson Type III distribution has fulfilled the requirements. Thus, the calculation of planned rainfall can use the Log Pearson Type III method. The example calculation for existing conditions with a 10-year return period is as follows.

Mean ($\bar{Z}$) = 2.11 mm
Standard Deviation (Sd_z) = 0.10
Variate Coefficient (C_V) = 0.05
Skewness Coefficient (C_S) = -0.98
Kurtosis Coefficient (C_K) = 4.66
K Value (K_Z) = 1.14

$$Z_T = \bar{Z} + K_Z \times Sd_z = 2.11 + 1.14 \times 0.10 = 2.23$$

$$Antilog\ Z_T = X_T = 167.93\ mm$$

The design rainfall obtained for the existing condition were 167.93 mm and for alternative conditions was 166.76 mm.

3.5 Hourly Rain Distribution

The average rainfall (R_t) and rainfall height (R'_t) were calculated from the first to the sixth hour, based on the optimum duration of rainfall for Makassar city, namely six hours [15].

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Table 5. Hourly Rainfall for 10 Years Return Period

Existing Condition			Alternative Condition		
t (hour)	**Rt (mm)**	**R't (mm)**	**t (hour)**	**Rt (mm)**	**R't (mm)**
1	92.414	92.414	1	91.771	91.771
2	58.217	24.020	2	57.812	23.853
3	44.428	16.850	3	44.119	16.733
4	36.674	13.414	4	36.419	13.321
5	31.605	11.328	5	31.385	11.249
6	27.988	9.902	6	27.793	9.833

Source: Analysis Result (2024)

3.6 Drainage System Modeling

A drainage network modeling of the Jongaya drainage system under existing and alternative conditions was conducted using the EPA SWMM 5.2 software.

1) Rain Gage
   Rain gage provides precipitation data for subcatchment areas within the study region, typically presented as rain distribution over time. Table 7 contains the data which will be entered into the Time Series.
2) Junction Node
   Junctions act as nodes in the drainage system, connecting channels and receiving subcatchment flows (outlets). The key parameter for junctions is the Invert Elevation, obtained from DEMNAS contour maps and adjusted by reducing the channel height.
3) Outfall Node
   Outfalls are the terminal nodes in a drainage system. The Jongaya drainage system has one outfall flowing directly to the sea. Parameters for outfall node O1 include invert elevation, obtained from DEMNAS contour maps, and type. The outfall type is Fixed, as it discharges into the sea under existing conditions.
4) Conduit
   Conduits, such as pipes or channels, transmit water between nodes. Secondary channels in the Jongaya system have open and closed rectangular cross-sections, based on data from Dinas Pekerjaan Umum Kota Makassar and field surveys. The primary Jongaya Canal's geometry input to SWMM uses cross-sectional data from Balai Besar Wilayah Sungai Pompengan Jeneberang. Conduit parameters include inlet and outlet nodes, shape, length, roughness, and offsets.
5) Subcatchments
   Subcatchments are hydrologic land units directing surface runoff to a single discharge point. In SWMM, subcatchments are manually digitized based on topography and flow direction data from existing tertiary channels.

- Area
  The area is determined by the SWMM software in hectares and automatically displayed.
- Width
  The width is measured from the furthest area beside the inlet junction to the channel.
- %Slope
  The slope of the subcatchment from the furthest point to the outlet (width). Calculating the slope of a subcatchment requires elevation data obtained from DEMNAS data.
- %Imperr
  %Imperv is obtained by digitizing impervious land using QGIS software and then calculating the percentage of the area to the total area of the subcatchment.
- N-Imperv
  The N-impervious, which represents the Manning coefficient applied to the impervious area of each subcatchment. The land in the Jongaya drainage system area has asphalt

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for the road and concrete for buildings. According to the SWMM 5.2 user manual, asphalt's manning coefficient is 0.011 while concrete is 0.012. Therefore, the average value of 0.0115 was used.

- N-Perv
  N-pervious is the manning coefficient applied to every subcatchment's pervious area [16]. The land in the Jongaya drainage system area has natural soil and short grass. According to the SWMM 5.2 user manual, natural cultivated soil's manning coefficient is 0.13 while short grass is 0.15. Therefore, the average value of 0.14 was used.
- Infiltration Data
  In this modeling, the curve number method is used for infiltration. The parameters needed for the curve number method are drying time and the curve number. The soil type in Makassar is dominated by ultisol which has a sandy loam texture [17]. Based on the SWMM 5.2 user's manual, it is found that the Saturated Hydraulic Conductivity (Ks) value of this type of soil is 0.57 in/hr and categorized into group B [18].
  $$T_{dry} = \frac{Ks}{\sqrt{0.02}} = \frac{0.57}{\sqrt{0.02}} = 4.03\ days$$
  The type of land use in most of the subcatchments in the Jongaya drainage system is dominated by green open spaces, roads and buildings. Based on the SWMM 5.2 user's manual, the curve number value for green open space is 61 and for roads and buildings is 98. The curve number value for each subcatchments can be calculated using the following formula:
  $$CN_{Subcatchment} = \frac{\Sigma(CN \times A_{land\ use})}{A_{subcatchment}}$$

6) Storage Unit
   In the alternative condition of the Jongaya drainage system, there is a long storage at the downstream end of the drainage system. The parameters are inverted elevation, max. the depth used an approach of 1.50 m obtained from a field survey and storage shape. The existing storage unit has an irregular shape; however, due to data limitations, a geometric approximation of a square shaped cross-section is used for the storage unit. The storage curve parameters are input in Tabular mode.
7) Flow Routing
   Initial flow routing aims to check whether the existing drainage system can handle the current high rainfall conditions. The method used for flow routing is Dynamic Wave.

- Existing Condition
  The flow routing reveals a persistent flooding with a volume of 169,521 m³. Conduit capacities are color-coded: dark blue (0-0.2), light blue (0.2-0.4), green (0.4-0.6), yellow (0.6-0.8), and red (0.8-1), with a value of 1 indicating flooding. Figure 2 shows several conduits in red, indicating they exceed 80% capacity.
- Alternative Condition
  After running the simulation, flooding with a volume of 148,477 m³ persists. Despite ongoing flooding, under alternative conditions, several conduits no longer exceed 80% of their capacity. Results are shown in Figure 3.
  Based on both flow routing results, an evaluation needs to be carried out so that flooding does not occur. The evaluation carried out was by adjusting the elevation and redesigning the channel dimensions. Pumps can also be necessary to be planned on the outlet of the storage unit in alternative conditions.

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Source: SWMM Simulation Result (2024)
Figure 2. Existing and Alternative Condition Flow Routing Schematic Result

3.7 Dimension and Elevation Adjustment

Conduit dimensions is determined by controlling the discharge, ensuring the hydraulic Q ≥ hydrologic Q by 0.001, and controlling the channel velocity (V) within the range of 0.30 – 2 m/s. The hydrologic Q will be calculated using the rational equation. The conduits to be redesigned are those whose existing dimensions can no longer accommodate runoff, based on the existing condition initial flow routing result.

1) Runoff Coefficient (C) Calculation
   Catchment area is classified into impermeable land (roofs and roads) and green open spaces (RTH) [19]. Runoff coefficients are based on surface properties and are derived from SNI 2415:2016 as shown in Table 8. The runoff coefficients for each type of land surface are converted into a composite runoff coefficient, $C_{Composite}$.

Table 6. Runoff Coefficient Value (C)

The Characteristics of Soil Surface	Runoff Coefficient Value, C
Highways and Roadside (Concrete)	0.75
Rooftops	0.75
Grass Yard, Sandy Soil	0.1

Source: Analysis Result (2024)

2) Concentration Time (T_C) Calculation
   Concentration Time (tc) is the time required for rainwater to flow from the furthest catchment point to the outlet. It includes the surface flow time to the nearest conduit inlet (t0) and the channel flow time to the outlet (tf) [20]. To calculate the t0 value, the manning roughness coefficient (nd) value is needed which is shown in Table 9.

Table 7. Manning Roughness Coefficient (nd)

Surface Type	Manning Roughness Coefficient (nd)
Rooftops	0.02
Street	0.013
Grass Yard	0.4

Source: Analysis Result (2024)

3) Rain Intensity Calculation
   The Mononobe method is used to calculate rainfall intensity. In this calculation, the rainfall intensity for all secondary and primary channels is determined using the 10-year return period rainfall data.
4) Q Hydrologic Calculation
   The hydrologic discharge (Q) in the study area is calculated using the Rational Method.

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The calculation results show that the highest surface runoff discharge in the secondary channel of both existing and alternative conditions is 19.851 m³/s. The total runoff in the downstream channel of the existing condition is 81.439 m³/s while on the alternative condition is 73.054 m³/s. These discharge results will be used for redesigning the drainage channels, which must be able to handle the planned flood discharge.

5) Conduit Dimension Analysis

The redesigned drainage channel uses U-Ditch type channels with covers, designed for urban areas where the channels will accommodate various types of vehicles. Dimensional analysis of the channels involves trial-and-error methods to determine hydraulic discharge (Q). As a result, the hydraulic discharge in the downstream channel in the existing conditons is 81.441 m³/s and on alternative conditions is 73.055 m³/s.

Based on the analysis results, a redesign of the dimensions was carried out for 34 secondary channels and 40 primary channels under existing conditions, and 34 secondary channels and 31 primary channels under alternative conditions.

6) Elevation Adjustment Analysis

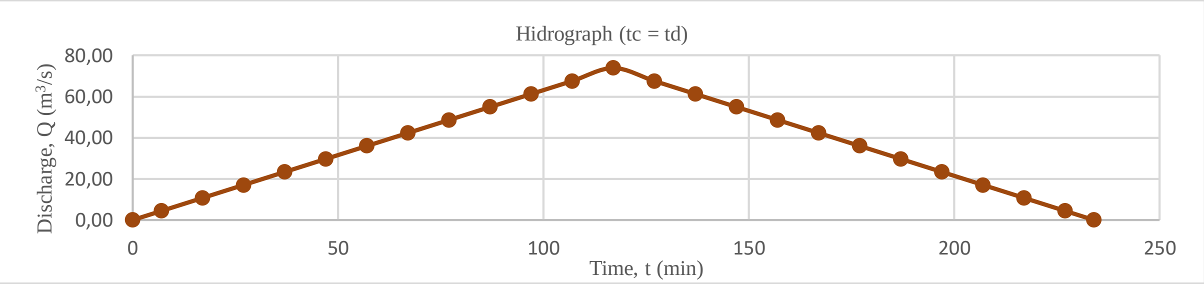
Elevation adjustments are also made to ensure the channels more effectively convey water to the outlet. This is done by readjusting the channel elevation according to the new dimensions from the redesign.

3.8 Storage Unit Evaluation

The Jeneberang Lama storage unit, with a 1,020,356 m² area and 3 m depth, allows for a 1.00 m water height and a maximum volume of 1,020,356 m³ after accounting for a 1.50 m freeboard. Channel C109, upstream of the storage unit, has a maximum inflow of 73.79 m³/s and a time of concentration of 117 minutes. Analyses are performed for td = tc, 237 minutes, and 357 minutes.

1) tc = td Condition

Based on the analysis result hydrograph shown in Figure 4, the maximum water level (h) in the storage unit is found to be 1.01 meters, with a final storage volume of 1,028,183.80 m³.



Source: Research Analysis (2024)

Figure 3. Storage Unit Hydrograph (tc = td)

2) td = 237 minutes Condition

The analysis for td = 237 minutes shows the storage unit reaching a 1.53-meter water level with a 1,559,471.80 m³ volume by 357 minutes. To prevent overflow with higher td values, a gate will be installed to manage outflow. A total of 2 sluice gates are planned. In this analysis, the gate opening height is adjusted according to the upstream water level. The storage unit analysis with td = 237 minutes includes the influence of the pump and sluice gate as its outflow calculation at minute 337 can be seen below.

n gate = 2
Gate width (B) = 1.50 m
Pillar width (b) = 0.50 m

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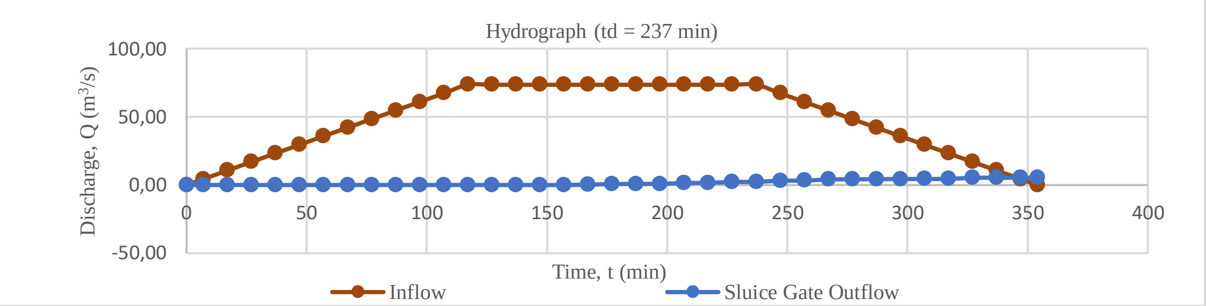
Gravity	= 9.81 m3/s
h1	= 0.62 m
Gate opening height (a)	= 0.70 m
Outflow (Q)	$= 0.8 \times 1.5 \times 0.7 \times \sqrt{2 \times 9.81 \times 0.62} = 2.68$ m3/s

Based on the planning, the gate opening height is determined to be 0.70 meters. In this scenario, it is assumed that td = 237 minutes and the gate starts operating at the 167th minute, where the gate begins to open to a height of 0.10 cm and the opening is gradually increased as the water level in the storage unit rises. The water level at the outlet is receding, allowing it to accommodate the runoff from the gate by gravity. The analysis results show that the water level at the 354th minute is 1.49 m with a storage volume of 1,523,717.61 m³. Therefore, the sluice gate can still operate and can be closed when the water level recedes to 0.9 m. The hydrograph shown on Figure 5.



Source: Research Analysis (2024)

Figure 4. Storage Unit Hydrograph (td = 237 min)

3) td = 307 minutes Condition

At td = 307 minutes, the storage unit reaches a 1.83-meter water level and 1,869,389.80 m³ volume by minute 424, exceeding the allowable level. To manage this, sluice gates and four pumps (each with 1.50 m³/s discharge) are planned to start from minute 167 to minute 424.

Q_{inflow}	= 35.95 m3/s
V_{inflow}	= 23,461.44 m3
Cummulative V_{inflow}	= 1,807,917.05 m3
$Q_{outflow}$	= 6 m3/s
$V_{outflow}$	$= \frac{1}{2} \times \Delta t \times (Q_{(n-1)} + Q_n) \times 60 = \frac{1}{2} \times (367 - 357) \times 6 + 6) \times 60$
	= 3,600 m3
Cummulative $V_{outflow}$	= 73,800 m3
$V_{final\ storage}$	= 1,693,632.42 m3
$h_{storage}$	$= \frac{V_{final\ storage}}{storage\ area} = \frac{1{,}693{,}632.42}{1{,}020{,}356} = 1.66$ m

Along with pumps, a total of 2 sluice gates are also planned with the same design as in the previous condition. In this scenario where both pumps and sluice gates operating, it is assumed that the water level at the outlet is receding, allowing it to accommodate the runoff from the gate by gravity.

The analysis at td = 307 minutes with pumps and sluice gates outflow shows that the maximum water level in the storage unit reaches 1.67 meters, with a final storage volume of 1,708,627.65 m³, as depicted on the hydrograph in Figure 6. The results indicate that the water level at the 424th minute exceeds the planned allowable water level and reaches the safety height. Therefore, in this scenario, both facilities, the pumps and sluice gates, need to continue operating until the water level decreases to 1.5 m, at which point the pumps can be turned off. Subsequently, when the water level reaches 0.9 m, the sluice gates can be closed again.

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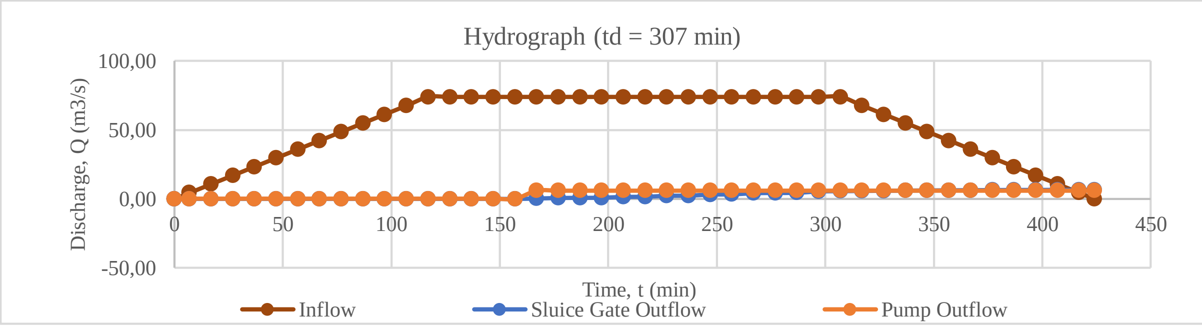
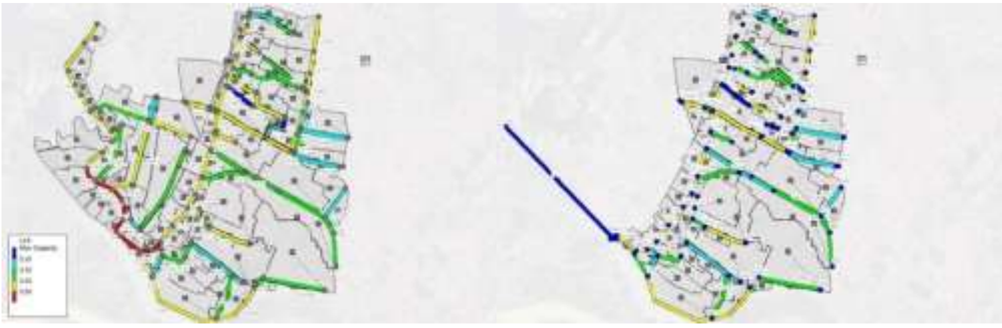


Source: Research Analysis (2024)
Figure 5. Storage Unit Hydrograph (td = 307 min)

3.9 Flow Routing for Evaluation

After redesigning the channel dimensions and adjusting elevations for both existing and alternative conditions, and conducting an evaluation of the storage unit along with adding drainage facilities such as pumps and sluice gates for alternative conditions, the results of the flow routing evaluation in both conditions show that flooding no longer occurs, as depicted in Figure 7. SWMM analysis showed the existing drainage system's flood discharge was 103.43 m^3/s, while the alternative was 77.48 m^3/s.



Source: SWMM Simulation Result (2024)
Figure 6. Existing and Alternative Condition Evaluation Flow Routing

4. Conclusion

The evaluation of the Jongaya Drainage System in Makassar, based on the analysis conducted, concludes that the system currently cannot handle rainfall runoff, leading to flooding. With a 10-year return period, the maximum flood discharge was 103.43 m^3/s for a rainfall intensity of 167.926 mm in the existing condition, while it decreased to 77.48 m^3/s for 166.759 mm in the alternative condition. Several channels in the drainage system have been redesigned with new cross-sectional dimensions. The storage unit, with an inflow discharge of 73.79 m^3/s, holds a capacity of 1,523,717.61 m^3 at minute 354, with a water level of 1.49 m. The system will be equipped with four pumps, each with a 1.5 m^3/s capacity, and two sluice gates measuring 1.50×0.80 m with a thickness of 20 mm.

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