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Hydrological, Hydraulic, and Topographic Analysis and Cost Estimation for Flood Control Design in Penajam Paser Utara Regency

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Abstract – Penajam Paser Utara Regency in East Kalimantan Province faces a high risk of flooding due to inadequate drainage channel capacity, irregular bed slopes, and sedimentation that reduces channel cross-sections. Increased runoff caused by land-use changes has exacerbated flooding, particularly in densely populated residential areas. This study aims to prepare a comprehensive Survey, Investigation, and Design (SID) document for flood control as a basis for planning measurable drainage infrastructure development. The research methods include collecting rainfall data from the Waru Station, calculating design flood discharges using the Gumbel Type I, Log-Pearson Type III, Log-Normal, and Frechet distributions, conducting hydraulic analyses using the Strickler and Manning formulas, simulating channel profiles with the Hydrologic Engineering Center – River Analysis System (HEC-RAS) software, and performing topographic surveys to determine channel cross-sections and benchmark elevations. The results indicate that the highest design flood discharge occurs in Waru Village, reaching 4.751 cubic meters per second for a 25-year return period. The hydraulic simulations support the need for channel normalization, slope adjustments, dimensional standardization, construction of polders, flap gates, and the operation of water pumps. The study concludes that drainage development should be implemented in stages with community participation in maintenance and pump operation funding.

Keywords: flood control, design flood discharge, topographic survey, channel hydraulics, cost estimation

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1. Introduction

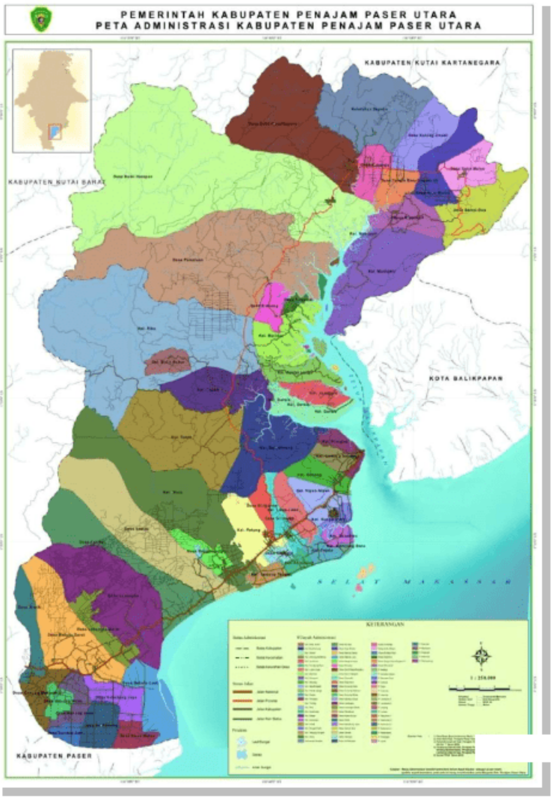
Penajam Paser Utara Regency in East Kalimantan Province frequently experiences flooding in densely populated residential areas such as Gunung Seteleng, Waru, Jalan Unocal, Penajam Indah Lestari Housing Complex, Linda Regency Housing Complex, Gang Masjid Al Ibroh, and Gang Hijrah. Flooding occurs due to the insufficient capacity of drainage channels, non-uniform channel bed slopes that create depressions impeding water flow, and sedimentation that reduces channel cross-sectional areas. The increase in runoff is further exacerbated by land-use changes that convert infiltration areas into residential zones. In response to these conditions, the East Kalimantan Provincial Government, through the Department of Public Works, Spatial Planning, and Public Housing, is implementing a comprehensive Survey, Investigation, and Design (SID) for Flood Control as the basis for planning measurable and phased drainage infrastructure development.

2. Methodology

The research methodology describes the location and implementation period, hydrological analysis methods, hydraulic analysis methods, topographic surveys, drainage channel design, and cost estimation planning. This methodology is designed to ensure that the resulting plan can be implemented according to actual field conditions.

2.1. Location and Period

The survey was conducted at six priority sites: Jalan Unocal Left, Jalan Unocal Right, Penajam Indah Lestari Housing Complex, Linda Regency Tujuh Housing Complex, Gang Masjid Al Ibroh, Gang Hijrah, and Waru Village. The activities were carried out during the 2023–2024 fiscal year.



Sumber: Rencana Tata Ruang Wilayah (RTRW) Kabupaten Penajam Paser Utara Tahun 2011 - 2031

Figure 1 Map of Penajam Paser Utara Regency

2.2. *Hydrological Analysis Method*

Hydrological analysis was conducted to calculate the design flood discharge for the designated sites, using three distribution approaches: the Gumbel Type I Distribution, the Log-Pearson Type III Distribution, and the Two-Parameter Log-Normal Distribution. Each distribution method is presented below along with its respective equations and explanations.

2.2.1 Gumbel Type I Distribution

The Gumbel Type I distribution is used to predict the design rainfall for a certain return period.

$$X = \overline{X} + (S \times K)$$

$$K = \frac{Y_T - Y_n}{S_n}$$

Description:

- **X** = Design rainfall (mm)
- **$\bar{X}$** = Mean rainfall (mm)
- **S** = Standard deviation
- **K** = Frequency factor
- **Y_T** = Reduced variate
- **Y_n** = Reduced mean
- **S_n** = Reduced standard deviation

2.2.2 Log Pearson Type III Distribution

The Log Pearson Type III distribution uses a logarithmic transformation approach.

$$\log X_T = \overline{\log X} + (G \times S)$$

$$S = \sqrt{\frac{\sum(\log X_i - \overline{\log X})^2}{n-1}}$$

$$CS = \frac{n\sum(\log X_i - \overline{\log X})^3}{(n-1)(n-2)S^3} \quad ; \quad CK = \frac{n^2\sum(\log X_i - \overline{\log X})^4}{(n-1)(n-2)(n-3)S^4}$$

Description:

- **log X_T** = Logarithm of design rainfall
- **log $\bar{X}$** = Mean of log rainfall
- **G** = Frequency factor
- **S** = Standard deviation of log rainfall
- **CS** = Skewness coefficient
- **CK** = Kurtosis coefficient

2.2.3 Two-Parameter Log Normal Distribution

The Two-Parameter Log Normal Distribution is used as additional validation.

$$\log X_T = \overline{\log X} + (K \times S_{\log X})$$

$$S_{\log X} = \sqrt{\frac{\sum(\log X_i - \overline{\log X})^2}{n-1}}$$

$$CV = \frac{\sigma}{\mu} \quad ; \quad CS = 3CV + CV^3 \quad ; \quad CK = CV^8 + 6CV^6 + 15CV^4 + 16CV^2 + 3$$

- **CV** = Coefficient of variation
- **σ** = Standard deviation of log X population
- **μ** = Mean of log X population
- **CS** = Skewness coefficient
- **CK** = Kurtosis coefficient

2.2.4 Frechet Method

The Frechet distribution, also called the Extreme Value Type II or Gumbel Type II distribution, is used to analyze hydrological data with extreme values. The cumulative probability is calculated using:

$$Y = a(\log X - X_0)$$

where:

- $a = 1.282\sqrt{\frac{1}{S_{\log X}}}$
- $X_0 = \log \overline{X} - 0.445(S_{\log X})$

- **log $\bar{X}$** = Mean logarithm of the observed data
- **Slog $\bar{X}$** = Standard deviation of the logarithm of the observed data
- **Y** = Gumbel reduced variate
- **CS** = Sample skewness coefficient
- **S** = Sample standard deviation
- **$\bar{X}$** = Sample mean
- **Xi** = Data value at i
- **CK** = Sample kurtosis coefficient

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2.3. Hydraulic Analysis Method

Hydraulic analysis is used to calculate the channel capacity based on velocity, discharge, and trapezoidal cross-section dimensions

2.3.1 Strickler's Formula

Open channel flow velocity is calculated using:

$$V = k \cdot R^{2/3} \cdot I^{1/2}$$

$$V = \frac{Q}{A} \quad ; \quad R = \frac{A}{P}$$

$$A = (b + mh)h \quad ; \quad A = (n + m)h^2$$

$$P = b + 2h\sqrt{m^2 + 1} \quad ; \quad P = (n + 2\sqrt{m^2 + 1})h$$

Description:

- **V** = Flow velocity (m/s)
- **Q** = Flow discharge (m³/s)
- **A** = Wetted cross-sectional area (m²)
- **P** = Wetted perimeter (m)
- **k** = Strickler coefficient
- **R** = Hydraulic radius (m)
- **I** = Channel bed slope
- **b** = Channel bottom width (m)
- **h** = Flow depth (m)
- **m** = Side slope of the channel
- **n** = Design parameter

2.3.2 Water Surface Profile Analysis

$$H_f = Sfx = \frac{1}{2}(S1 + S2) \times x$$

$$H_1 = Z_1 + \alpha_1 \frac{V_1^2}{2g} \quad ; \quad H_2 = Z_2 + \alpha_2 \frac{V_2^2}{2g}$$

$$Z_1 + \alpha_1 \frac{V_1^2}{2g} = Z_2 + \alpha_2 \frac{V_2^2}{2g} + hf + he$$

$$He = k\left(\alpha \frac{V^2}{2g}\right)$$

Description:

- **Z** = Channel bed elevation (m)
- **V** = Flow velocity (m/s)
- **g** = Acceleration due to gravity (m/s²)
- **α** = Energy coefficient
- **hf** = Friction head loss
- **he** = Local head loss

The water surface profile simulation is carried out using HEC-RAS.

2.4. Topographic Survey

A topographic survey is carried out to collect ground surface data, elevation benchmarks, channel cross-sections, and site features for design purposes.

2.5. Drainage Channel Design

The technical design of the channel is adjusted to the design flood discharge, with an average cross-section width of one meter, a maximum depth of 1.2 meters, equipped with flap gates and a polder system.

2.6. Cost Estimate (RAB)

The Cost Estimate is prepared based on the work volume: earthworks, stone masonry, concrete casting, HDPE pipes, sluice gates, and water pumps.

3. Results and Discussion

This chapter presents the results of the surveys and technical calculations, covering hydrological analysis, hydraulic analysis, topographic surveys, drainage channel design, cost estimation, and the implementation strategy for flood control in Penajam Paser Utara Regency

3.1. Hydrological Survey and Analysis

3.1.1 Data Availability

Rainfall data were obtained from the Waru Station with a recording period of ten years. This data represents all planned sections because the area is located within a single micro-watershed with a uniform rainfall pattern.

Table 1 Recapitulation of Monthly Rainfall Data at Waru Station, 2012–2021

Tahun	Bulan												Jumlah
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	
2012	146.00	139.20	368.20	237.40	75.60	212.00	55.00	41.00	89.50	159.50	282.00	326.50	2131.90
2013	282.20	127.30	212.30	112.20	158.40	210.50	109.50	98.50	20.00	129.50	182.50	120.00	1762.90
2014	228.00	371.00	180.70	195.00	180.50	145.00	240.00	70.30	50.00	210.00	230.00	260.00	2360.50
2015	210.00	120.00	140.00	270.00	190.00	160.00	50.00	150.00	10.00	70.00	70.00	370.00	1810.00
2016	220.00	130.00	170.00	240.00	260.00	220.00	20.00	0.00	0.00	0.00	0.00	0.00	1260.00
2017	429.00	231.00	223.00	264.00	214.00	150.00	222.00	105.00	152.00	184.00	194.00	249.00	2617.00
2018	250.00	40.00	280.00	215.00	290.00	200.00	432.00	211.00	170.00	162.00	415.00	180.00	2845.00
2019	281.00	325.00	432.00	178.00	242.50	278.00	190.50	65.50	63.10	128.50	106.00	422.60	2712.70
2020	380.60	148.50	154.00	231.50	164.20	576.70	31.00	94.10	67.70	98.50	198.80	211.80	2357.40
2021	160.20	179.90	235.00	320.80	203.20	458.10	335.80	178.50	254.90	278.30	250.20	165.00	3019.90
Rata-rata	258.70	181.19	239.52	226.39	197.84	261.03	168.58	101.39	87.72	142.03	192.85	230.49	2287.73

Figure 2 Annual Maximum Daily Rainfall at Waru Rainfall Station

3.1.2 Design Rainfall Data Analysis

The design rainfall data analysis was conducted using several probability distribution methods. The table below presents the Chi-Square and Smirnov–Kolmogorov test results for each distribution to determine the best fit for the design rainfall calculation.

Table 2 **Recapitulation of Design Rainfall by Each Method**

No.	Return Period (years)	Probability distribution			
		Gumbel I (mm)	Log Normal 2 Parameter (mm)	Log Pearson III (mm)	Frechet (Gumbel II) (mm)
1	2	112.1986	112.5193	114.0642	110.6756
2	5	142.5094	136.4682	137.0135	133.3927
3	10	162.5778	151.4572	150.0547	145.9378
4	20	181.8927	165.3286	162.2404	157.8523
5	25	187.9343	183.5942	164.7487	177.2328
6	50	206.7452	197.2913	174.7112	192.8279
7	100	225.4171	196.0555	183.9023	229.729

Based on the Chi-Square and Smirnov–Kolmogorov tests, the design rainfall used for further calculations will be based on the Log Pearson III method, as it shows the best fit with the observed data.

3.1.3 Design Flood Discharge Analysis

Prior to presenting the calculated design flood discharges for each return period, this section provides a list of the areas and channel names that are the objects of the analysis. This list is intended to facilitate location identification and maintain clarity in the subsequent design discharge tables.

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Table 3 Summary of Regions and Channel Names

Region	Channel Name
[1]	[2]
Jalan Unocal Kiri	UCL.Ki
Jalan Unocal Kanan	UCL.Ka
Perumahan Penajam Indah Lestari	PPIL
Perumahan Linda Regency 7	PLR
Gang Masjid Al Ibroh Penajam	MAIP
Gang Al Hijrah	AH
Desa Waru	WR

The design flood discharge is calculated using the Rational Method based on the design rainfall data. The flood discharge values for each return period—5 years, 10 years, 20 years, and 25 years—are presented in the following tables.

Table 4 Design Flood Discharge Calculation for 5-Year Return Period

Length (L) (km)	Slope	Area (A) (km^2)	Tc (Jam)	I (mm/ jam)	c	Q (m^3/s)
[5]	[6]	[7]	[8]	[9]	[10]	[11]
0.6	0.003	0.044	25.164	137.01	0.5	0.846
0.6	0.003	0.02	25.164	137.01	0.5	0.385
1.62	0.004	0.228	49.054	137.01	0.4	3.471
1.31	0.003	0.112	43.536	137.01	0.4	1.709
0.532	0.003	0.044	22.646	137.01	0.4	0.667
1.191	0.002	0.052	48.192	137.01	0.4	0.792
2.603	0.004	0.313	64.919	137.01	0.3	3.573

Table 5 **Design Flood Discharge Calculation for 10-Year Return Period**

Length (L) (km)	Slope	Area (A) (km2)	Tc (Jam)	I (mm/ jam)	c	Q (m3/s)
[5]	[6]	[7]	[8]	[9]	[10]	[11]
0.6	0.003	0.044	25.164	150.1	0.5	0.927
0.6	0.003	0.02	25.164	150.1	0.5	0.422
1.62	0.004	0.228	49.054	150.1	0.4	3.802
1.31	0.003	0.112	43.536	150.1	0.4	1.872
0.532	0.003	0.044	22.646	150.1	0.4	0.731
1.191	0.002	0.052	48.192	150.1	0.4	0.868
2.603	0.004	0.313	64.919	150.1	0.3	3.914

Table 5 Design Flood Discharge Calculation for 20-Year Return Period

Length (L) (km)	Slope	Area (A) (km^2)	Tc (Jam)	I (mm/ jam)	c	Q (m^3/s)
[5]	[6]	[7]	[8]	[9]	[10]	[11]
0.6	0.003	0.044	25.16	162.2	0.5	1.002
0.6	0.003	0.02	25.16	164.7	0.5	0.463
1.62	0.004	0.228	49.05	162.2	0.4	4.109
1.31	0.003	0.112	43.54	162.2	0.4	2.024
0.532	0.003	0.044	22.65	162.2	0.4	0.79
1.191	0.002	0.052	48.19	162.2	0.4	0.938
2.603	0.004	0.313	64.92	162.2	0.3	4.23

Table 6 **Design Flood Discharge Calculation for 25-Year Return Period**

Length (L) (km)	Slope	Area (A) (km2)	Tc (Jam)	I (mm/ jam)	c	Q (m^3/s)
[5]	[6]	[7]	[8]	[9]	[10]	[11]
0.600	0.003	0.044	25.164	164.749	0.500	1.018
0.600	0.003	0.020	25.164	164.749	0.500	0.463
1.620	0.004	0.228	49.054	164.749	0.400	4.174
1.310	0.003	0.112	43.536	164.749	0.400	2.055
0.532	0.003	0.044	22.646	164.749	0.400	0.802
1.191	0.002	0.052	48.192	164.749	0.400	0.953
2.603	0.004	0.313	64.919	164.749	0.300	4.297

3.1.4 Domestic, Small-Scale Industry, and Service Wastewater Discharge Analysis

The calculation of domestic, small-scale industrial, and service wastewater discharges refers to the regional and channel classification presented in Table 3. This approach ensures consistency with the catchment areas analyzed in the previous design flood discharge calculations.

Table 7 **Design Discharge Calculation for 5-Year Return Period (Q5)**

Q Rainwater	Q Wastewater	Q Design
(m^3/s)	(m^3/s)	(m^3/s)
[3]	[4]	[5]
0.846	0.161	1.008
0.385	0.064	0.449
3.471	0.069	3.54
1.709	0.328	2.038
0.667	0.064	0.731
0.792	0.076	0.868
3.573	0.455	4.028

Table 8 **Design Discharge Calculation for 10-Year Return Period (Q10)**

Q Rainwater	Q Wastewater	Q Design
(m^3/s)	(m^3/s)	(m^3/s)
[3]	[4]	[5]
0.927	0.161	1.088
0.422	0.064	0.486
3.802	0.069	3.871
1.872	0.328	2.200
0.731	0.064	0.794
0.868	0.076	0.943
3.914	0.455	4.368

Table 9 **Design Discharge Calculation for 20-Year Return Period (Q20)**

Q Rainwater	Q Wastewater	Q Design
(m^3/s)	(m^3/s)	(m^3/s)
[3]	[4]	[5]

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1.002	0.161	1.163
0.463	0.064	0.527
4.109	0.069	4.179
2.024	0.328	2.352
0.790	0.064	0.854
0.938	0.076	1.013
4.230	0.455	4.685

Table 10 **Design Discharge Calculation for 25-Year Return Period (Q25)**

Q Rainwater	**Q Wastewater**	**Q Design**
(m^3/s)	**(m^3/s)**	**(m^3/s)**
[3]	[4]	[5]
1.018	0.161	1.179
0.463	0.064	0.527
4.174	0.069	4.243
2.055	0.328	2.384
0.802	0.064	0.866
0.953	0.076	1.028
4.297	0.455	4.751

3.2. Topographic Survey and Analysis

The topographic survey produced data on ground surface conditions, natural features (rivers, channels), and man-made structures (roads, bridges, settlements). The data were analyzed to support flood control planning through detailed situation maps, cross sections, longitudinal sections, and inundation area boundaries.

3.2.1 Bench Mark Installation and Coordinate Data

Eight Bench Marks were installed at strategic points within the study area. Each Bench Mark measures 20 × 20 × 100 cm, is painted blue for easy identification, and is placed on stable ground. The Bench Marks serve as reference points for the horizontal and vertical control framework.

The measured coordinates are shown in Table 11 below.

Table 11 **BM CP**

No	**Patok**	**X**	**Y**	**Z**
1	BM1	474133.906	9861708.263	4.179
2	BM2	458405.327	9847171.592	1.694
3	BM3	474136.519	9861628.059	3.580
4	BM4	474382.804	9860064.067	2.949
5	BM5	456463.194	9846098.445	2.575
6	BM6	456448.426	9846110.990	2.488
7	BM7	458483.770	9847202.952	1.316
8	BM8	458370.517	9847611.032	1.696

3.2.2 Channel Alignment, Polder Site, and Inundation Area Mapping

The channel alignment survey was conducted from the polder site towards the Mahakam River, with cross-section stakes installed every 50 meters (25 meters in bends). The polder site measurement covered an area of ±0.3–7 hectares using spot height methods, while the inundation area survey supported flood elevation mapping for identifying flood-prone areas.

3.2.3 Horizontal and Vertical Control

Polygon measurement as the horizontal control was tied to the reference Bench Marks, ensuring accurate map positioning. Levelling was conducted on polygon points and cross-section stakes to ensure map elevations match actual site conditions.

3.2.4 Survey Result Visualization

The survey results are visualized through the river cross section (Figure 3) and the combined longitudinal section and situation map (Figure 4) to illustrate the river profile and drainage network. The drainage channel location is documented in Figure 5, while the flood conditions near settlements are shown in Figure 6.

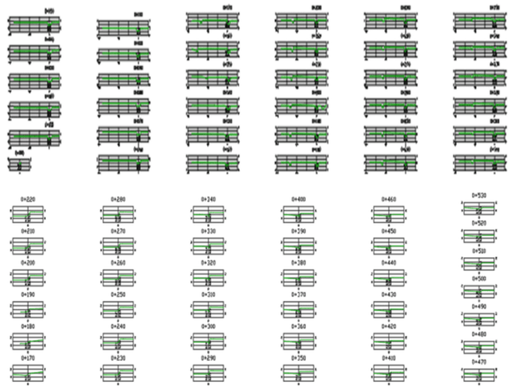

Figure 3 River Cross Section

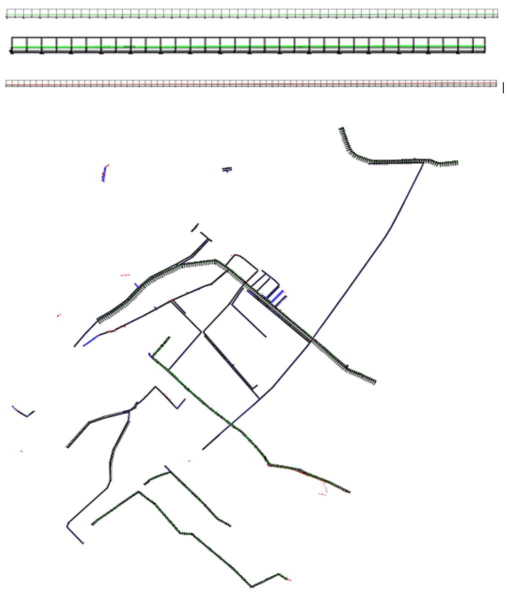

Figure 4 Situation Map and Longitudinal Section.

Figure 5 Drainage Channel Location

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Figure 6 Flood Condition at the Study Site

3.3. Hydraulic Analysis and Drainage Planning

The hydraulic analysis is carried out to evaluate the capacity of the drainage channels in accommodating the peak discharge obtained from the hydrological analysis. The design discharge values Q5, Q10, Q20, and Q25 are derived from the hydrological analysis in Sub Bab 3.1. This analysis aims to ensure that the channel capacity is adequate and supports the planning of drainage improvements and new channel construction to minimize potential flooding in residential areas.

3.3.1 Water Surface Profile Basis

The water surface profile calculation uses the Standard Step Method with HEC-RAS software to model the water surface profile in open channels. The Manning and Strickler formulas are applied to calculate flow velocity, discharge, energy loss, and channel bed slope. This calculation supports the simulation results explained in the next subchapter. The relationship of bed elevation, energy loss, and water surface profile is shown in Figure 7

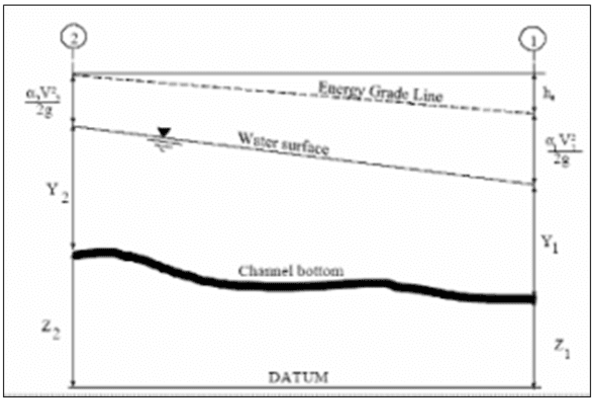

Figure 7 Energy Equation Diagram

3.3.2 Water Surface Profile Simulation Result

The water surface profile simulation was carried out for several priority channel segments in the study area, namely Unocal Street, Indah Lestari Housing, Linda Regency, Masjid Ibroh Alley, Hijrah Alley, and Waru. All segments were analyzed using the same method and parameters to obtain an overview of the channel capacity in both existing and planned conditions.

As an example, are presented below in Figures 8 and 9, showing the longitudinal profile and the cross sections at STA 640 and STA 540. The simulation results for other segments can be found in the supporting technical report.

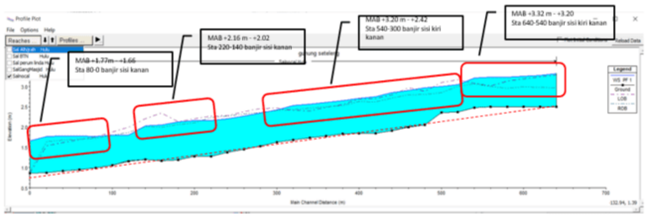

Figure 8 Unocal Channel Longitudinal Section

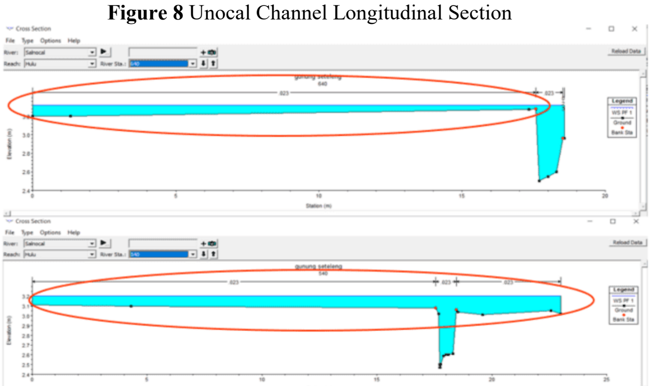

Figure 9 Unocal Channel Cross Section STA 640 & STA 540

3.3.3 Drainage Plan Sketch

As part of the flood control plan, the drainage network in the Gunung Seteleng Zone and Waru Zone is designed with a main channel, polder, pumps, and flap gates to manage the flood discharge. The channel alignment sketches are presented in Figures 10 and 11.

Figure 10 Gunung Seteleng Drainage Plan Sketch

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Figure 11 Waru Drainage Plan Sketch

3.4. Drainage Channel Design

The drainage channel design aims to adjust the channel capacity to the peak discharge from the hydraulic simulation. The parameters considered include channel bed width, depth, bed slope, flow velocity, and water surface elevation. In low-lying areas, a polder system with pumps and flap gates is used to ensure that the flood discharge remains controlled even when the river water level is higher than the drainage channel.

3.4.1 Channel Dimension Calculation Result

The channel sizing is based on calculations using the Manning and Strickler formulas, with validation performed through HEC-RAS simulations. Table 3.2 provides a summary of the planned dimensions, including bed width, depth, bed slope, and design discharge.

Table 12 Planned Channel Dimensions for Q25

No	Channel Name	Channel Code	Q (m^3/s)	Q calculated (m^3/s)	Check	Side Slope (1 : m)	I	k	n = b/h	V (Q/A) m/s	b (m)	h (m)	A (m^2)	P (m)	R	Proposed Dimensions (b × h) (m)
1	Jalan Unocal Left	UCL.Ki	1.179	1.179	0	1:01	0	60	1	1.227	1.2	0.8	0.96	4.02	0.24	1.200 × 1.000
2	Jalan Unocal Right	UCL.Ka	0.527	0.528	0	1:01	0	60	1	0.989	1	0.53	0.53	3.07	0.17	1.000 × 2.000
3	Penajam Indah Lestari Housing Left	PPIL.Ki	2.122	2.121	0	1:01	0	60	1	1.577	1.2	1.12	1.35	4.64	0.29	1.200 × 1.200
4	Penajam Indah Lestari Housing Right	PPIL.Ka	2.122	2.121	0	1:01	0	60	1	1.577	1.2	1.12	1.35	4.64	0.29	1.200 × 1.200
5	Linda Regency 7 Housing Left	PLR.Ki	1.176	1.176	0	1:01	0	60	1	1.221	1.2	0.77	0.92	3.97	0.25	1.200 × 1.000
6	Linda Regency 7 Housing Right	PLR.Ka	1.176	1.176	0	1:01	0	60	1	1.221	1.2	0.77	0.92	3.97	0.25	1.200 × 1.000
7	Masjid Al Ibroh Alley	MAIP	0.866	0.866	0	1:01	0	60	1	1.156	1	0.75	0.75	3.5	0.21	1.000 × 1.000
8	Al Hijrah Alley	AH	1.028	1.028	0	1:01	0	60	1	1.19	1	0.81	0.81	3.57	0.22	1.000 × 1.000
9	Waru Village Left	WR.Ki	2.376	2.376	0	1:01	0	60	1	1.758	1.3	1.04	1.35	4.68	0.29	1.300 × 1.200
10	Waru Village Right	WR.Ka	2.376	2.376	0	1:01	0	60	1	1.758	1.3	1.04	1.35	4.68	0.29	1.300 × 1.200

3.5. Cost Estimation

The Budget Plan (RAB) is prepared based on the estimated work volumes calculated from the approved design drawings. The estimation includes preparation works, excavation, masonry/concrete works, backfilling, and supporting works such as pipe installation, pumps, and paving. The total cost is calculated using the 2023 Unit Price Analysis for Labor and Materials (HSUB).

Table 12 Summary of Drainage Construction Budget Plan

No	Lokasi Saluran / Channel Location	Total Biaya / Total Cost (Rp)
1	Jalan Unocal	7,221,340,000
2	Perumahan Penajam Indah Lestari	14,589,306,000
3	Perumahan Linda Regency 7	16,356,146,000
4	Gang Masjid Ibroh	3,995,524,000
5	Gang Al Hijrah	7,132,164,000
6	Waru	28,127,185,000
Total		**77,421,665,000**

4. Conclusion and Recommendations

4.1. Conclusion

1. Flooding in Penajam Paser Utara Regency is primarily caused by drainage channels that are inadequately dimensioned drainage channels that are inadequately dimensioned to accommodate the discharge.
2. In addition, the inconsistent slope of the channels creates depressions that obstruct smooth water flow.
3. Sedimentation problems further reduce the channels' storage capacity.
4. Repairs of damaged channels are needed to ensure smooth water flow.
5. A substantial budget is required for the construction, so it would be better to implement it in stages, based on each channel, with intervals between construction phases.
6. The average dimensions of the channels are approximately 1.00 m wide with a maximum depth of 1.20 m.
7. Standardizing dimensions is necessary to ensure adequate storage capacity, allowing the channels to function as long storage to handle tidal backflow at the river mouth.
8. Pump operation is needed during high tide season, which requires operators and operational costs.

4.2. Recommendations

1. Repairs should be carried out gradually for each channel.
2. There should be initiatives from the local community and relevant agencies to maintain the channels, including routine dredging to reduce sedimentation.
3. Local residents should be aware of the need to operate and fund the pumps, for example through community contributions for fuel supply or electricity costs for pump operation.

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