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Evaluation of Drainage System on the Samarinda – Bontang Axis Road, East Kalimantan

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***Abstract*—** *The Samarinda–Bontang axis road is the main route connecting cities in East Kalimantan to APT. Pranoto International Airport and connecting to Bontang and Tanjung Santan oil refineries. At STA 0+000 (Muara Badak three way juction) to STA 4+250 it often experiences flooding which can hamper transportation activities. The cause of flooding is from changes in land use and the existing condition of the channel that is too small. The purpose of this study was to evaluate the capacity of the drainage channel on the Samarinda-Bontang axis road. Calculation of flood discharge of 5-year return period using the Nakayasu SUH method and flow modelling using the HEC-RAS application program. The 5-year design flood discharge for each channel is obtained as follows: the Right 1 was 2.323 m³/s, the Right 2 was 4.211 m³/s, the Right 3 was 12.469 m³/s, the Left 1 was 11.163 m³/s, the Left 2 was 14.759 m³/s and the Left 3 was 22.934 m³/s. It was concluded that Right Channel 1 did not overflow while the drainage channels, Right 2, Right 3, Left 2, and Left 3 overflowed. Because the dimensions of the new design channel are much larger, it is necessary to design other flood management alternatives such as the creation of reservoirs and/or infiltration wells.*

***Keywords*—** *Drainage channel; Evaluation; HEC-RAS; Samarinda-Bontang.*

I. INTRODUCTION

Intercity activities of Samarinda as the provincial capital and Bontang as one of the industrial cities in East Kalimantan Province are growing, especially with the opening of flight paths at Aji Pangeran Tumenggung Pranoto Airport, starting May 24, 2018, which is located on the Samarinda – Bontang axis road. The hilly and valley topography of the area makes some locations on this axis road vulnerable to inundation due to rain. Changes in the land use, especially mining activities in the vicinity, contribute to increasing runoff discharge to the side ditch.

The field survey showed that the road section between Muara Badak three way junction to Samarinda, at STA 0+00 - STA 4+250, is a valley area and there are often puddles when it rains. The dimensions of the existing channel are in the form of U-ditch with a width of 80 cm and a height of 100 cm. The road consists of 2 lanes with a width of 5.5 m and a road shoulder of 1.5 m. The road pavement is rigid pavement coated with asphalt.

The purpose of this study was to evaluate the capacity of the drainage channel on the Samarinda – Bontang Axis Road (starting from Muara Badak three way junction to APT. Pranoto Samarinda). Flow modeling used the Hydrologic Engineering Center - River Analysis System (HEC-RAS) application to determine the flow capacity of drainage channels by looking at the flood water level that occurs. Where hydrological analysis as input in modeling using Nakayasu flood hydrograph. The channel dimensions used are based on the largest capacity comparison between the existing channel discharge and the designed flood discharge.

II. LITERATURE REVIEW

A. Hydrological Analysis

Hydrological analysis is intended to obtain the hydrological and meteorological characteristics of the Channel Drainage Area. (Triatmodjo B., 2010). Hydrological analysis carried out includes design rainfall analysis, determination of channel catchment areas, rainfall intensity, and design flood discharge.

Design rainfall is an estimate of rain that will occur in a watershed with a certain return period, for example 2, 5, 10, or 20 years. According to (Wesli, 2008) the use of return-periods for drainage channel planning is:

1. Quarter Channel : 1 year return period
2. Tertiary Channel : 2 year return period
3. Secondary Channel : 5 years return period
4. Primary Channel : 10 years return period

Rainfall intensity is the amount of rain mass that falls and is expressed in terms of the volume of rain per unit of time. If the existing rain data is only daily rain data, then the intensity of rain can be calculated by the following Mononobe formula. (Suripin, 2003):

$$I_t = \frac{R_{24}}{t}\left(\frac{24}{t}\right)^{\frac{2}{3}}$$

Where:

I_t : Rainfall intensity for the duration of rain t (mm/hour)
t : Duration of precipitation (hours)
R_{24}: Maximum rainfall for 24 hours (mm)

A catchment area is a plain bounded by ridges or topographic boundaries to receive, store and drain rainwater that falls on it into river channels and flows into tributaries and main rivers, which then gather together into lakes, rivers or seas.

Analysis of design discharge on drainage channels can be carried out using rational methods or hydrographs. By calculating the channel discharge that occurs can use the guidelines of urban drainage channel design standards and technical design standards.

TABLE I. Hydrological Design Criteria of Urban Drainage Systems

Area (ha)	Return Period (T) Year	Method
<10	2	Rational
10 - 100	2-5	Rational
101 - 500	5-20	Rational
>500	10-25	Unit Hydrograf
(Suripin, 2004)		

The calculation of the Nakayasu Synthetic Unit Hydrograph requires the following characteristics of the flow area parameters: The longest length of the main river flow; Catchment area; Time base of hydrograph; Time period from rain surface to hydrograph peak (time of peak), and Time period from the heavy point of rain to the heavy point of the hydrograph (time lag). (Soemarto, 1987)

The calculation is as follow:

$$Q_P = \left(\frac{C.A.R_0}{3{,}6(0{,}3T_P + T_{0,3})}\right)$$

Where (Fig.1):

- Q_P : Peak flood discharge (m^3/s)
- R_0 : Unit precipitation (1/mm)
- T_P : Time period from rain surface to hydrograph peak (hour)
- $T_{0,3}$: Time required for discharge to decrease from peak to 30% from peak discharge (hour)
- A : Watershed area (km^2)
- C : run off coefficient

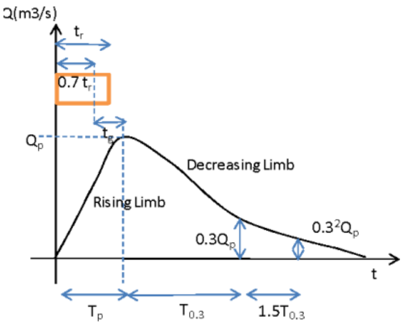

Fig. 1. Nakayasu Synthetics Unit Hydrogaph
(A.B. Safarina, et.al., 2011)

Next, calculating the design discharge for various recharges with the following equation: (Sri Harto, 1993)

$$Qk = U_1R_1 + U_2R_{1-1} + U_3R_{1-2} + \ldots U_nR_{1-(n-1)} + Bf$$

Where:

- Qk : Flood hydrograph ordinate at 1st hour
- Un : Unit hydrograph ordinate
- R1 : Net rain at 1st hour
- Bf : Base flow

B. Hydraulical Analysis

Hydraulics analysis serves to determine channel capacity by taking into account the properties of hydraulics that occur in drainage channels as follows: types of flow (steady or unsteady), channel material roughness figures, flow properties (critical, subcritical and supercritical).

Channel cross-section can be said to be economical if the channel cross-section that has a minimum wet circumference is able to provide maximum capacity to the channel cross-section (Peraturan Menteri PU No. 12, 2014).

The rectangular channel cross-section (Fig.2) is most economical when the width of the channel base is twice the depth of water (B=2h) or the hydraulic radius is half of the water depth (R=h/2).

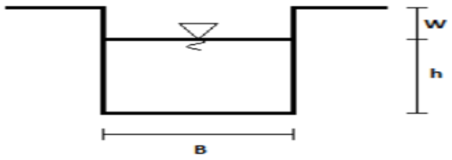

Fig. 2. Rectangular Drainage

And the free board is $w = \sqrt{0{,}5 \text{ x } h}$.

C. HEC-RAS

HEC-RAS is an application program to model flows in the River Analysis System (RAS) river. HEC-RAS can perform one-dimensional count analysis on permanent flow water table profile (Steady Flow), one/two-dimensional count on Unsteady Flow water level profile, sediment transport count, water quality analysis, and hydraulic design features. HEC-RAS is an application program that integrates graphical user interface features, hydraulic analysis, data management and storage, graphics, and reporting (Istiarto, 2014).

III. Research Methodology

The research begins with field observations in the form of flow directions, road conditions, channels, and the use of the surrounding area. Then, collecting the support data, namely daily rainfall, topography map and the land use. Design rainfall calculations from rainfall data of the last 11 years and analyzed using E.J. Gumbell, Log Normal and Log Pearson III distributions tested for suitability with Chi-Square and Smirnov-Kolmogorov Test and also statistical parameter test. Design flood discharge was calculated using Nakayasu Synthetic Unit Hydrograph. Then evaluate the capacity of the existing channel against the designed flood discharge. And the channels with a capacity smaller than the designed flood discharge are re-designed and simulated with the HEC-RAS. The stages on evaluation of drainage system on the Samarinda-Bontang axis road as shown in Fig. 3.

A. Research Sites

The research location is on the Samarinda-Bontang axis road, Kutai Kartanegara, East Kalimantan Province. This location is showed in Fig. 4 below.

B. Maximum Daily Rainfall

Based on observations of rainfall that occurred at the Sungai Siring Post, maximum daily rainfall data for 2012-2022 was obtained in Table II.

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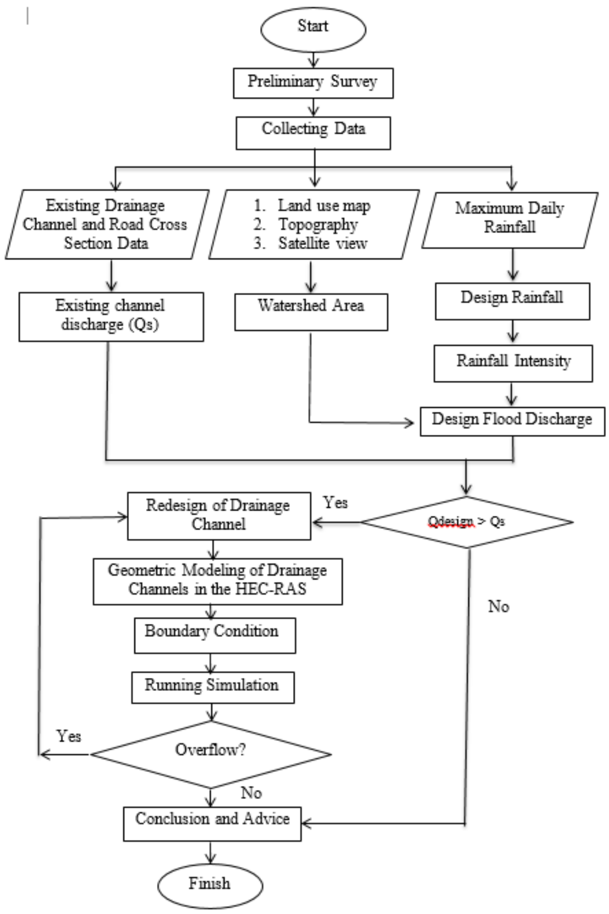

Fig. 3. The Research Flow on Evaluation of Drainage System on the Samarinda-Bontang Axis Road

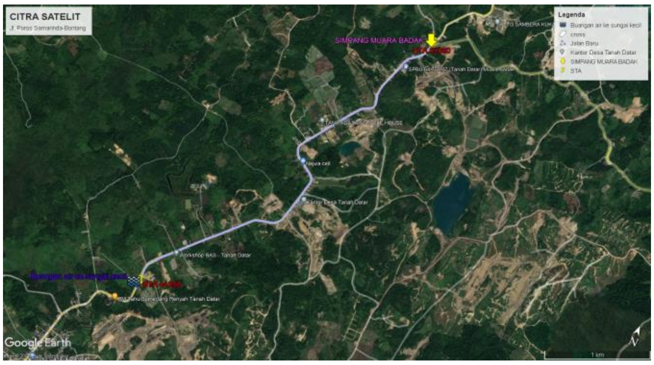

Fig. 4. Samarinda – Bontang Axis Road (Google Earth, 2023)

TABLE II. Maximum Daily Rainfall at Sungai Siring Post

Year	Max. Daily Rainfall (mm)	Year	Max. Daily Rainfall (mm)
2012	53.90	2018	78.70
2013	128.50	2019	120.60
2014	103.50	2020	109.40
2015	63.00	2021	214.50
2016	133.20	2022	108.60
2017	90.60		

(Balai Wilayah Sungai Kalimantan IV Samarinda-Kalimantan Timur)

C. *The Land Use*

In Fig. 5 the following are the results of land mapping on the Samarinda-Bontang axis road.

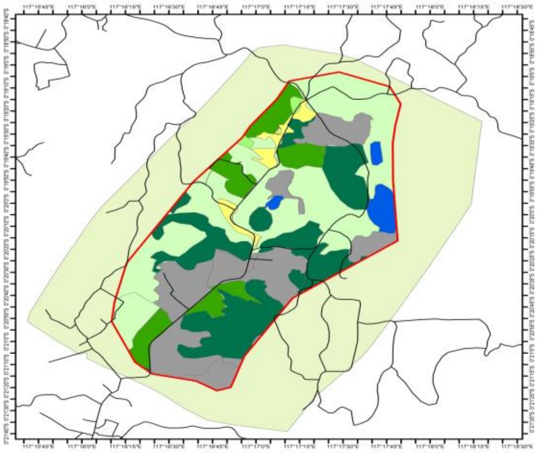

Fig. 5. The Land Use of Samarind-Bontang Axis Road
(Badan Perencanaan Pembangunan Daerah Kota Samarinda)

IV. Result And Discussion

A. *Design Rainfall*

The results of rainfall calculations using the E. J. Gumbel Method, Normal Log Method and Log Person III Method are presented in the Table III below.

TABLE III. Design Rainfall

No.	Return Period (Year)	Design Rainfall (mm)		
		E.J. Gumbel	Log Normal	Log Pearson III
1	2	103,557	102,544	102,544
2	5	154,167	140,950	140,771
3	10	187,676	166,507	167,117

The results of parametric tests on rain design using all three methods are shown in the Table IV below.

TABLE IV. Parametric Test Results

Method	Criterion	Result	
E. J. Gumbel	Ck ~ 5.4	3,581	Not eligible
	Cs ~ 1.14	0,981	
Log Normal	$Ck = Cv^8 + 6\ Cv^6 + 15\ Cv^4 + 16\ Cv^2 + 3$	2,374	Not eligible
	$2{,}374 \neq 3{,}108$		
	$Cs = Cv^3 + 3\ Cv$	0,057	
	$0{,}057 \neq 0{,}246$		
Log Pearson III	Ck = Bebas	2,374	Eligible
	Cs = Bebas	0,057	

The results of non-parametric tests using the Smirnov-Kolmogorov test and Chi-Square test are summarized in Table V as follows.

TABLe V. Non Parameter Test Results

Method	Smirnov - Kolmogorov	Result (Δ max)		Chi - Square	Result (X^2)	
	Δ max < Δ kr			$X^2 < X^2$ Kritis		
E. J. Gumbel	Δ max < 0.40	0,133	Eligible	$X^2 <$ 5.991	1,3	Eligible
Log Normal	Δ max < 0.40	0,091	Eligible	$X^2 <$ 5.991	1,3	Eligible
Log Pearson Tipe III	Δ max < 0.40	0,099	Eligible	$X^2 <$ 5.991	4,9	Eligible

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Based on the both distribution tested for suitability, it can be seen that the appropriate distribution for the rain data is Log Pearson III. Then the design rain used in the next calculation is the 5 year return period design rain based on Log Pearson III, which is 140.77 mm.

B. The Catchment Area

Catchment Area is a rainfed area where water flowing on the surface is accommodated by existing channels. Calculation of flow discharge by dividing the catchment area into segments on the right and left of the road. There are 3 segments on the right side and 3 segments on the left with each land use as shown in the following figure

Fig. 6. The Catchment Area

Based on the catchment area and land use in each flow segment, the runoff coefficient and the effective rain can be calculated. The area of the flow segment, the value of the runoff coefficient, and the effective rain are shown in the table below.

TABLE VI. The Runoff Coefficient and Effective Rain

The Channel	Watershed Area (km2)	Runoff Coefficient, C	Effecvtive Rain, Re (mm)
Right 1	0.31	0.35	49.66
Right 2	1.09	031	40.05
Right 3	2.62	0.32	45.65
Left 1	2.19	0.27	37.81
Left 2	3.73	0.31	43.71
Left 3	4.80	0.36	50.55

C. Design Flood Discharge

Effective rain in the previous calculation needs to find the distribution of the hours to calculate the design flood discharge. The pattern of rain distribution in Indonesia ranges from 4-7 hours every day and in this study it was taken 5 hours. The effective rain distribution of the hours is calculated using the Mononobe method. (Rahmani, R.N., 2016)

The results of the distribution of each hour effective rain for each channel (Right 1, Right 2, Right 3, Left 1, Left 2, and Left 3) are summarized in the following table below.

Nakayasu synthetic unit hydrographs are made for each flow segment. Furthermore, the flood discharge of each channel is calculated based on the Nakayasu synthetic unit hydrograph and the effective rain distribution every hour. And the results of the calculation of flood discharge design for the 5 year return period are shown in Figure 7 and Figure 8 below.

TABLE VII. The Effective Rain of Each Hour

t (h)	I_t of R_{24} (mm/h)	R_t of R_{24} (mm)	Effective Rain of Each Hour (mm)					
			R1	R2	R3	L1	L2	L3
0,00	0,000	0,000	0.00	0.00	0.00	0.00	0.00	0.00
0,50	0,928	0,464	23.05	18.59	21.19	17.55	20.29	23.46
1,00	0,585	0,121	5.99	4.3	5.51	4.56	5.70	6.10
1,50	0,446	0,085	4.20	3.39	3.86	3.20	3.70	4.28
2,00	0,368	0,067	3.35	2.70	3.08	2.55	2.95	3.41
2,50	0,317	0,057	2.83	2.28	2.60	2.15	2.49	2.88
3,00	0,281	0,050	2.47	1.99	2.27	1.88	2.17	2.51
3,50	0,254	0,044	2.21	1.78	2.03	1.68	1.94	2.25
4,00	0,232	0,040	2.01	1.62	1.84	1.53	1.77	2.04
4,50	0,215	0,037	1.85	1.49	1.70	1.41	1.62	1.88
5,00	0,200	0,035	1.71	1.38	1.58	1.30	1.51	1.74
Total		1,000	49.66	40.05	45.65	37.81	43.71	50.55

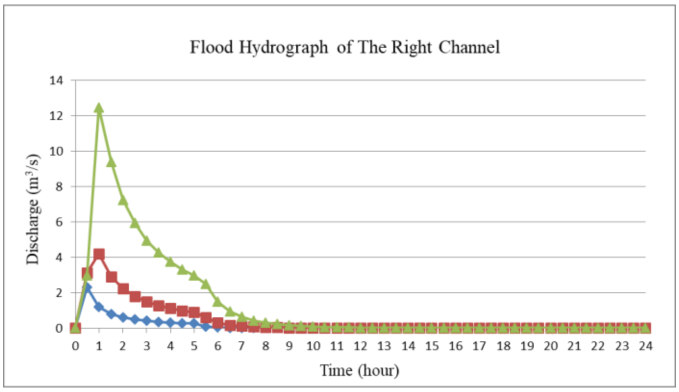

Fig. 7. Flood Hydrograph of The Right Channel

Where the peak flood discharge of each channel is the Right 1: 2.323 m^3/s, the Right 2: 4.211 m^3/s, the Right 3: 12.469 m^3/s, the Left 1: 11.163 m^3/s, the Left 2: 14.759 m^3/s, and the Left 3: 22.934 m^3/s.

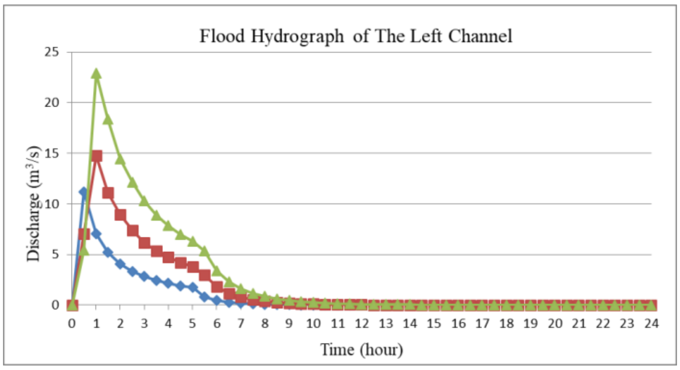

Fig. 8. Flood Hydrograph of The Left Channel

D. Evaluation of Drainage System

This evaluation is needed to determine the channel's ability to drain the flood discharge. The calculation of the existing channel capacity was carried out first, then compared with the flood discharge designed for the 5-year return period. If the design flood discharge is greater than the discharge that the existing channel can pass, then the channel needs to be redesigned.

The capacity of existing channels is as follows: Right Channel 1: 2.375 m^3/s, Right 2: 1.611 m^3/s, Right 3: 1.475 m^3/s, Left 1: 2.273 m^3/s, Left 2: 1.708 m^3/s, and Left 3: 1.639

m^3/s. Based on the results of calculations show that only the right channel 1 can drain flood discharge. Other channels have a small capacity of flood discharge, so it can be concluded that the water is overflowing.

For this reason, a redesign of the channel dimensions was carried out to be able to drain flood discharge. The redesigned channel dimensions are shown in the following table.

TABLE VIII. The New Channel Dimensions

Channel	STA	Width B (m)	Waterdepth h (m)	Height H (m)	Slope, S
Right 1	0+000-1+000	0.80	0.60	1.00	0.0079
Right 2	1+000-2+725	1.68	0.84	1.49	0.0110
Right 3	2+725-4+250	2.88	1.44	2.29	0.0055
Left 1	0+000-1+225	2.78	1.36	2.19	0.0059
Left 2	1+225-3+125	3.14	1.57	2.45	0.0050
Left 3	3+125-4+250	3.91	1.96	2.95	0.0037

And then, the design flood discharge was simulated on the new channel dimensions using HECRAS. The display of the simulation results of the example on the Right Channel 2 is presented in Figure 9 and Figure 10 below.

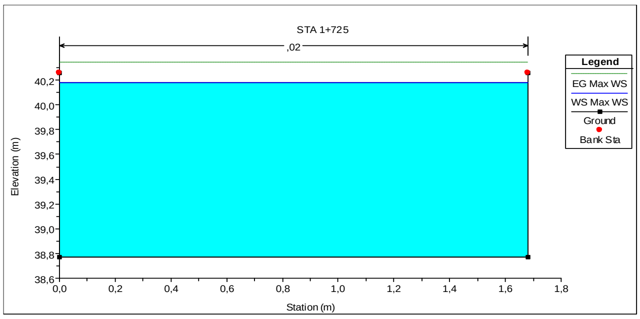

Fig. 9. Channel Cross Section of the Right 2

The Figure 9 shows that as a result of running HEC-RAS in the Right 2, water does not overflow. The flood water level is below the channel height of 0.01 m. And the Figure 10 shows the long section of HEC-RAS running results on the Right 2 with a channel length of 1,725 m.

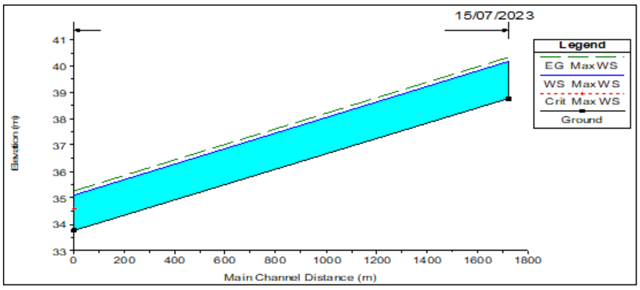

Fig. 10. Channel Long Section of the Right 2

Based on the simulations that have been carried out, the redesigning of the new drainage channel can drain the discharge with a 5-year return period. However the dimensions of the channel required are much larger than the land provided fo the channel. So it is necessary to plan other flood prevention alternatives such as retarding basin and /or infiltration wells.

V. Conclusion

Based on the results of the evaluation of the exixting channels, it was found that in general, the channels are not capable of carrying discharge at a return period of 5 years. Redesign of the channel dimensions resulted in much larger dimensions. For this reason, alternative flood management solutions are needed other than changing the dimensions of the channel, such as making retarding basin and /or infiltration wells.

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