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Original Article

Analysis of Water Level of Binjeita River for Several Return Periods of Flood

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Abstract *- Binjeita river is located at Binjeita village of Bintauna sub-district of North Bolaang Mongondow District, Indonesia. Due to several upcoming bridge construction projects on the Binjeita river requires information about the water elevation of the Binjeita river to complete the base elevation of a bridge. Furthermore, flood water elevation will determine the potential overflow during inundation along the Binjeita river. This investigation aims to obtain the water elevation of flood at Binjeita river, especially at the river segment where Bridge 1 is located. To do so, secondary data such as rainfall data from the rain station of Sangkub-Huntuk and discharge data from Andagile-Tontulow station are required. These two stations are located in the Binjeita river basin. The rainfall data will be analysed to determine its quality, followed by calculating average rainfall using the Thiessen Polygon method. The rain frequency is analysed using the Log Pearson Type III. The HSS-SCS method, with the aid of HEC-HMS, is used to analyse the design discharge. The result is as follows: at a return period of 25, 50 and 100 years, the discharge is 2 m3/sec, 2.5 m3/sec and 3 m3/sec consecutively around Bridge 1. No overflow takes place around Bridge 1 as a result of discharge at those return periods. However, upstream and downstream, overflow occurs with various water elevations where 50 cm is the maximum water elevation.*

Keywords *- Discharge, Water elevation, HSS-SCS, HEC-HMS, Thiessen Polygon.*

1. Introduction

The Binjeita river is located at Binjeita village of the Bintauna sub-district of North Bolaang Mongondow district. As a new district, North Bolaang Mongondow district is developing itself, where infrastructures are the primary concern in the development program. The PLTU Sulawesi I (electric steam power plant) is a project that takes place in the district, and the exact location is next to the Binjeita river. PLTU Sulawesi I will utilise the Binjeita river's water as its power plant's source to provide electricity throughout the district. The project requires bridges to cross the Binjeita river and its streams, where the bridge design will consider the water elevation of the Binjeita river when a flood occurs at a specific return period. This has become the primary concern and will be considered as the purpose of the research.

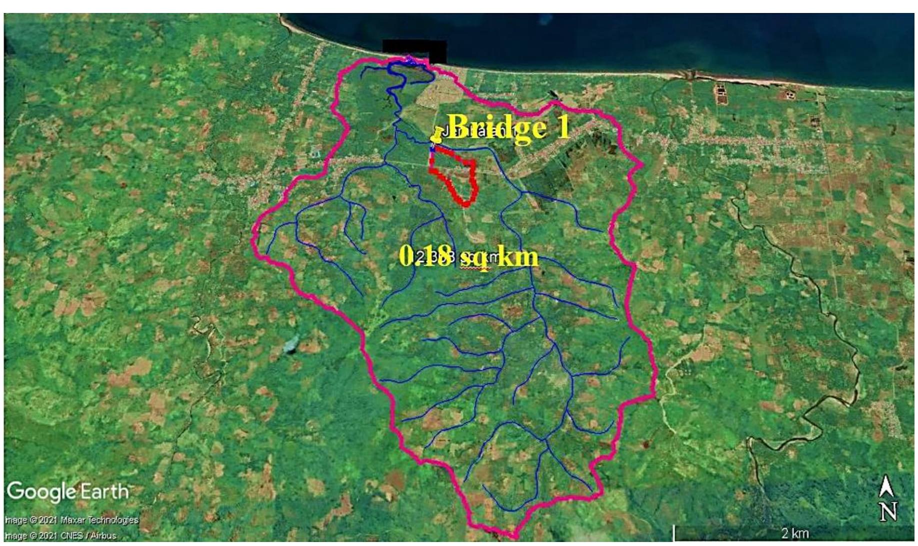

Fig. 1 Area of Research

2. Literature Review

Triatmodjo (2009) stated that frequency analysis aims to ascertain the probability of an event and the frequency or return period of such an event using a probability distribution. The statistical parameters in hydrology analysis are mean, standard deviation, coefficient of variance, coefficient of skewness and coefficient of kurtosis.

The probability distribution describes a likelihood of variance as the substitute for its frequency. One purpose of the probability distribution analysis is to determine the return period. The return period is a hypothetic time where discharge or rain with a certain quantity will be equalised or exceeded once at a certain period. The function of probability distribution in this research are:

1) Normal Distribution,
2) Log-Norma Distribution,
3) Gumbel Distribution, and
4) Log-Pearson Type III Distribution.

The Soil Conservation Service (SCS, 1972, in Chow, 1988) developed a method to calculate adequate rainfall from heavy rain in the form below:

$$P_e = \frac{(P-0.2S)^2}{P+0.8S} \quad (1)$$

Where:
Pe = Effective precipitation (mm)
P = Precipitation
S = Maximum potential retention, mostly by infiltration

The above equation is a fundamental equation to determine precipitation. The maximum potential retention is calculated by the equation below:

$$S = \frac{25400}{CN} - 254 \quad (2)$$

CN is the curve number to take total precipitation for various river basin characteristics with different soil types and land use into account (Supit, 2013). The design discharge is the maximum discharge of a river at a specific return period. The features to determine design discharge are rainfall data, catchment area and land cover data. Several methods usually obtain design discharge of flood. This research will apply the empirical method of unit hydrograph to compute flood discharge.

The non-dimensional SCS (Soil Conservation Services) is a synthetic unit hydrograph where discharge is specified as the ratio of discharge q to peak discharge q_p and time as the ratio of time t to time to peak of unit hydrograph t_p. Should peak discharge and time of recession of a duration of adequate rain (lag time) be informed, the unit hydrograph can be estimated from the synthetic unit hydrograph of SCS.

Lag Time (Tl) is calculated as follows:

$$Tl = \frac{L^{0.8}(2540-22.86\,CN)^{0.7}}{14.104\,CN \times s^{0.5}} \quad (3)$$

Time to peak is expressed as follows:

$$Tp = \frac{Tr}{2} + Tl \quad (4)$$

Peak discharge (q_p) is calculated as follows:

$$qp = \frac{2.08 * A}{Tp} \quad (5)$$

Steady flow is a flow where its velocity is constant for a while. Natural flow is generally unsteady, caused by the channel's hydraulic geometry, irregular river pattern, plants along the river slope, hydraulic structure, the transformation of the river bed, etc. The steady flow component determines the water surface profile at a steady flow. Such components can model the water surface profile at sub-critical, supercritical, and a combination of both.

Water surface profiles are computed from one cross-section to the next by solving the energy equation with an iterative procedure called the standard step method. The energy equation is written as follows:

$$Y_2 + Z_2 + \frac{\alpha_2 V_2^2}{2g} = Y_1 + Z_1 + \frac{\alpha_1 V_1^2}{2g} + he \quad (6)$$

Where:
Y_1, Y_2 = depth of water at cross sections Z_1
Z_2 = elevation of the main channel inverts V_1
V_2 = average velocities (total discharge/total flow area)
α_1, α_2 = velocity weighting coefficients
g = gravitational acceleration
h_e = energy head loss

3. Research Methodology

3.1. Analysis of Rainfall Data

The rainfall data is collected from the Sangkub-Huntuk rain station, the only station available around the river basin. The data itself is rainfall measurements from 2008 to 2019.

Table 1. Maximum Data of Daily Rainfall

Year	Maximum Daily Rainfall (mm) at Sangkub-Huntuk Rain Station
2008	80.83
2009	50.6
2010	80.6
2011	170.1
2012	302.5
2013	80.7
2014	90.6
2015	50.7
2016	61
2017	90.5
2018	79
2019	129

Source: River Basin Organization of Sulawesi I

3.2. Data Quality Test

It is necessary to test the quality of hydrology data before analysing it. The quality test will apply the outlier test to distinguish extensive deviation data. The outlier data test is based on the skewness coefficient (C_{Slog}).

3.3. Outlier Data Test of Sangkub-Huntuk Rain Station

Table 2. Statistical Parameter of Sangkub-Huntuk Rain Station

Slog =	0,2216
C_{Slog} =	1,2145

Source: Result of Analysis

The allowable upper margin is as follows:

$$Log\ Xh = \overline{LogX} + Kn * Slog \quad (7)$$

Log Xh = 2.44
Xh = 272.5 mm

The allowable lower margin is as follows:

$$Log\ Xl = \overline{LogX} - Kn * Slog \quad (8)$$

Log Xl = 1.51
Xl = 32.05 mm

3.4. Analysis of Average Rainfall at River Basin

Since there is only one station in the river basin area, the average rainfall is rainfall data from the Sangkub-Huntuk station.

Table 3. Average Rainfall Data at River Basin

Year	Average rainfall (mm)
2008	80,83
2009	50,6
2010	80,6
2011	170,1
2012	272,51
2013	80,7
2014	90,6
2015	50,7
2016	61
2017	90,5
2018	79
2019	129

Source: Result of Analysis

3.5. Frequency Analysis of Rainfall Data

Frequency analysis aims to obtain the design rainfall by using four common frequency distributions in hydrology analysis, which are:

1. Normal Distribution
2. Log-Normal Distribution
3. Gumbel Type 1 Distribution
4. Log Pearson Type 3 Distribution

Table 4. Statistical Parameter

Cs =	2,0931
Cv =	0,0450
Ck =	1,1706

Source: Result of Analysis

The analysis presented in Table 5 exposed the Log Pearson Type III distribution that will be applied to calculate the design rainfall since data distribution satisfied the Log Pearson Type III condition.

3.6. Log Pearson Type III Distribution

The Log Pearson Type III distribution is formulated as follows:

$$\log X_{TR} = \overline{\log X} + S_{log}.K_{TR,Cs} \quad (9)$$

Where:
$\log X_{TR}$ = log series of design rainfall
$\log \overline{X}$ = log series of average data
S_{log} = log series of deviation standard
$K_{TR,Cs}$ = frequency factor

S_{log} = 0,212
C_{slog} = 1,066
$\log \overline{X}$ = 1,959

Design rainfall (mm) for the return period of 25, 50 and 100 years are presented in the table below.

3.7. Analysis of Design Flood

Analysis of design flood will utilise HEC-HMS software. The Synthetic Unit Hydrograph of SCS is used to transform rain to flow and calculate the losses, while the Recession Constant method calculates base flow.

Table 5. Condition of Statistical Parameter

Type of Distribution	Data		Condition		Conclusion
	Cs	Ck	Cs	Ck	
Normal	2,0931	1,1706	0	3	Not Satisfy
Log-Normal			0.2751	4.299	Not Satisfy
Gumbel			1,14	5,4	Not Satisfy
Log Pearson Type III			Other than 3 Conditions above		Satisfy

Source: Result of Analysis

Table 6. Design Rainfall

Return Period (years)	Design Rainfall (mm)
25	209,6
50	248,1
100	283,4

Source: Result of Analysis

Table 7. Percentage of Rainfall Distribution

Hours	1	2	3	4	5	6	7
Percentage of Rainfall Distribution	22	28	19	15	7	6	3

Source: Sumarauw et.al, 2016

Table 8. Design of Hourly Rainfall

Return Period (mm)	Design Rainfall (mm)	Hour of Rainfall						
		1	2	3	4	5	6	7
5	137,1	30,16	38,39	26,05	20,57	9,60	8,23	4,11
10	170,2	37,43	47,64	32,33	25,52	11,91	10,21	5,10
25	209,6	46,12	58,70	39,83	31,44	14,67	12,58	6,29
50	248,1	54,59	69,47	47,14	37,22	17,37	14,89	7,44
100	283,4	62,35	79,35	53,84	42,51	19,84	17,00	8,50

Source: Result of Analysis

3.8. Transformation of Daily Rainfall to Hourly Rainfall

The transformation of daily rainfall to hourly rainfall will utilise the hourly rainfall pattern of the North Coast of Bolaang Mongondow district (Sumarauw et al., 2016).

By utilising the hourly rainfall pattern, the transformation of daily rainfall to hourly rainfall is presented in the table 8.

3.9. Analysis of Discharge using HEC-HMS

The HEC-HMS software is a common software in hydrology analysis. It is used to analyse design discharge in this research. The first step of running HEC-HMS is to calibrate its parameter.

Table 9. Comparison of River Basin Area

River Basin	Area (km^2)
Binjeita (at estuary)	12.368
Andegile (at measurement discharge station)	174

Source: Result of Analysis

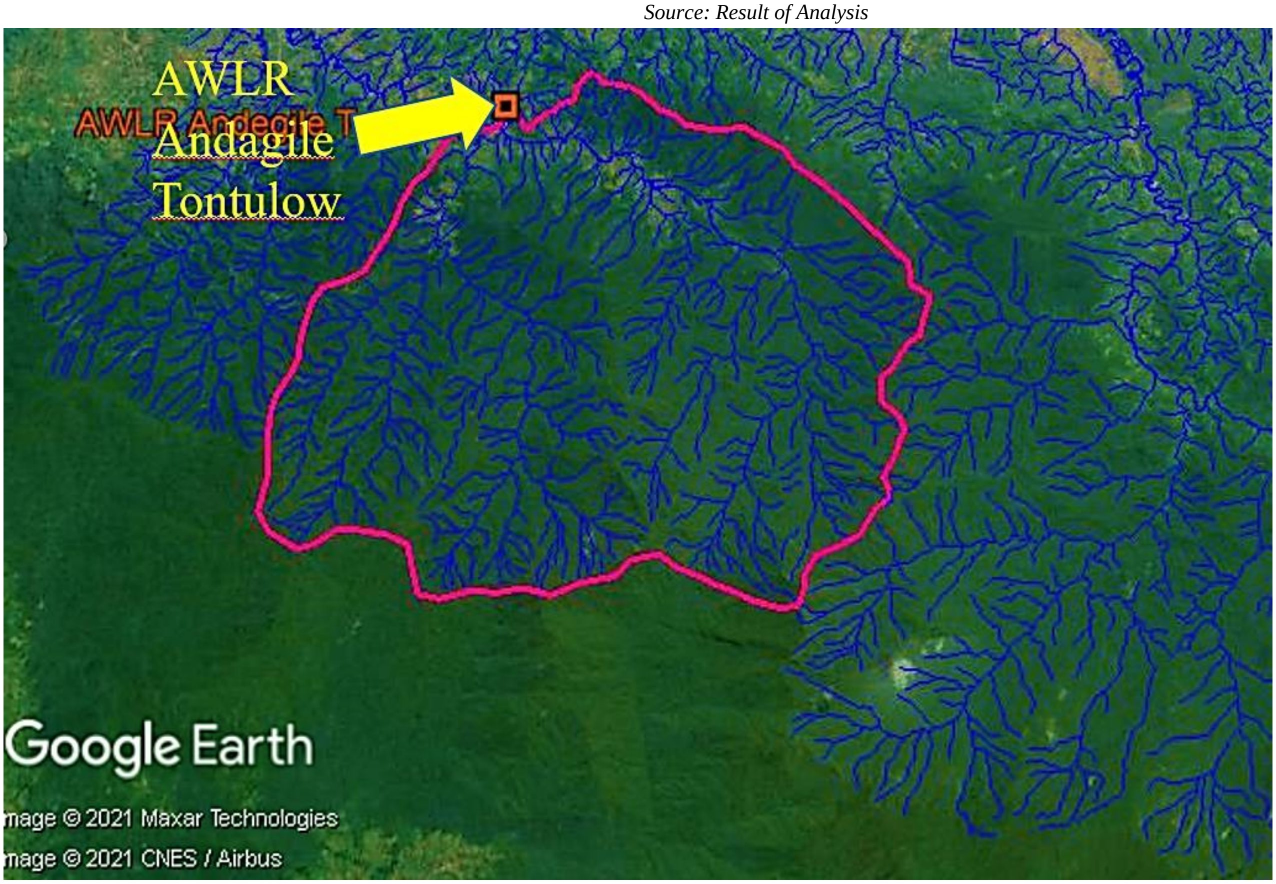

Fig. 2 Location of Discharge Measurement Station

Average rainfall data, sub-system and river basin area are analysed with HEC-HMS by entering river basin parameters according to HEC-HMS conditions.

3.10. Discharge Data

The discharge data will utilise the measured discharge data from the adjacent river with similar characteristics since there is no discharge measurement station located at Binjeita river. The Andegile-Tontulow station will provide the measured discharge data as it is located adjacent to the Binjeita river.

The record of discharge measurement of the Andegile river is presented in Table 10. The data in Table 10 is discharge data measured in 2013, where the peak of discharge occurred in the year 2013.

Estimation of discharge of Binjeita river at its estuary is obtained by using a comparison of river basin area method. Table 11 presents the result.

Table 10. Andagile river discharge data in 2013

DAY	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nop	Dec
1	34,63	8,97	8,35	3,74	8,65	6,67	8,65	3,74	2,45	2,45	5,93	6,42
2	32,06	16,79	4,08	3,74	10,32	5,48	2,84	7,47	4,84	2,00	3,91	11,43
3	18,93	13,92	4,08	3,91	18,38	5,93	4,84	7,20	7,47	3,91	3,42	7,47
4	12,22	13,92	3,91	3,74	9,29	7,20	4,84	8,35	4,84	2,84	2,98	5,48
5	11,05	13,92	3,91	8,35	12,22	7,47	3,74	4,45	3,91	2,58	6,17	3,91
6	11,05	18,38	4,64	8,05	8,65	8,05	32,06	3,74	3,58	1,53	4,64	3,91
7	15,30	17,84	3,42	5,70	6,67	5,70	16,28	3,74	4,45	1,53	2,98	5,05
8	12,63	16,79	2,84	4,64	17,84	5,48	11,43	4,26	3,74	1,45	3,42	3,58
9	12,63	11,05	4,26	5,48	12,63	6,93	6,93	4,45	2,84	1,45	2,84	3,42
10	19,50	9,97	4,64	4,64	12,22	6,17	10,68	3,27	2,45	1,37	2,45	3,27
11	13,92	9,97	3,74	4,26	11,43	7,20	14,83	3,42	3,42	1,29	2,22	3,27
12	11,82	8,65	3,74	5,70	16,28	5,48	9,97	6,93	3,91	1,37	2,22	3,12
13	8,97	13,05	3,74	5,05	10,68	5,05	8,65	4,08	5,70	1,29	2,11	2,84
14	8,35	8,65	3,74	3,91	106,69	9,29	7,75	3,42	5,93	1,22	2,11	4,26
15	15,30	20,67	3,74	5,93	71,20	6,67	8,05	3,12	10,32	1,02	2,11	3,12
16	10,68	5,93	2,98	4,45	20,08	5,70	8,35	3,27	6,93	0,84	2,33	2,98
17	8,97	6,17	2,98	20,67	11,82	5,48	9,97	2,58	5,48	0,84	2,71	3,27
18	12,63	5,93	8,05	15,30	22,53	4,45	8,97	2,98	5,48	1,29	4,26	3,58
19	12,63	5,26	5,26	3,74	20,67	3,91	8,05	2,98	3,74	1,15	3,91	3,12
20	8,97	4,45	4,64	8,05	13,92	3,91	20,08	2,98	3,42	1,15	3,58	2,58
21	12,63	3,42	3,27	5,70	13,48	3,58	16,28	3,91	2,71	1,29	2,71	3,42
22	11,82	7,47	5,93	4,45	13,05	3,42	11,43	3,91	2,58	1,29	5,48	5,26
23	9,63	5,26	5,70	5,70	10,32	3,27	8,05	3,91	2,45	1,29	5,26	3,74
24	8,97	5,48	8,97	6,67	16,28	3,12	6,17	14,83	2,45	1,29	4,26	5,93
25	8,65	6,42	6,67	12,22	23,84	2,98	5,93	2,33	2,22	1,45	3,42	4,84
26	7,75	7,75	7,75	7,47	13,05	3,27	5,48	2,33	2,22	1,22	2,84	4,08
27	222,66	7,75	4,08	69,70	10,68	2,84	3,91	4,45	2,00	1,62	2,63	5,05
28	31,23	5,93	3,74	11,05	9,63	3,91	3,74	3,58	2,11	1,37	3,58	4,64
29	17,30	0,74	3,74	13,48	8,97	2,84	6,17	17,30	2,00	1,71	5,26	4,45
30	12,22		3,27	9,63	12,22	2,58	3,74	2,71	1,90	9,29	3,58	3,27
31	8,65		3,12		8,65		2,84	2,33		5,93		5,93

Source: Result of Analysis

Table 11. Binjeita river discharge in 2013

DAY	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nop	Dec
1	2,46	0,64	0,59	0,27	0,62	0,47	0,62	0,27	0,17	0,17	0,42	0,46
2	2,28	1,19	0,29	0,27	0,73	0,39	0,20	0,53	0,34	0,14	0,28	0,81
3	1,35	0,99	0,29	0,28	1,31	0,42	0,34	0,51	0,53	0,28	0,24	0,53
4	0,87	0,99	0,28	0,27	0,66	0,51	0,34	0,59	0,34	0,20	0,21	0,39
5	0,79	0,99	0,28	0,59	0,87	0,53	0,27	0,32	0,28	0,18	0,44	0,28
6	0,79	1,31	0,33	0,57	0,62	0,57	2,28	0,27	0,25	0,11	0,33	0,28
7	1,09	1,27	0,24	0,41	0,47	0,41	1,16	0,27	0,32	0,11	0,21	0,36
8	0,90	1,19	0,20	0,33	1,27	0,39	0,81	0,30	0,27	0,10	0,24	0,25
9	0,90	0,79	0,30	0,39	0,90	0,49	0,49	0,32	0,20	0,10	0,20	0,24
10	1,39	0,71	0,33	0,33	0,87	0,44	0,76	0,23	0,17	0,10	0,17	0,23
11	0,99	0,71	0,27	0,30	0,81	0,51	1,05	0,24	0,24	0,09	0,16	0,23
12	0,84	0,62	0,27	0,41	1,16	0,39	0,71	0,49	0,28	0,10	0,16	0,22
13	0,64	0,93	0,27	0,36	0,76	0,36	0,62	0,29	0,41	0,09	0,15	0,20
14	0,59	0,62	0,27	0,28	7,58	0,66	0,55	0,24	0,42	0,09	0,15	0,30
15	1,09	1,47	0,27	0,42	5,06	0,47	0,57	0,22	0,73	0,07	0,15	0,22
16	0,76	0,42	0,21	0,32	1,43	0,41	0,59	0,23	0,49	0,06	0,17	0,21
17	0,64	0,44	0,21	1,47	0,84	0,39	0,71	0,18	0,39	0,06	0,19	0,23
18	0,90	0,42	0,57	1,09	1,60	0,32	0,64	0,21	0,39	0,09	0,30	0,25
19	0,90	0,37	0,37	0,27	1,47	0,28	0,57	0,21	0,27	0,08	0,28	0,22
20	0,64	0,32	0,33	0,57	0,99	0,28	1,43	0,21	0,24	0,08	0,25	0,18
21	0,90	0,24	0,23	0,41	0,96	0,25	1,16	0,28	0,19	0,09	0,19	0,24
22	0,84	0,53	0,42	0,32	0,93	0,24	0,81	0,28	0,18	0,09	0,39	0,37
23	0,68	0,37	0,41	0,41	0,73	0,23	0,57	0,28	0,17	0,09	0,37	0,27
24	0,64	0,39	0,64	0,47	1,16	0,22	0,44	1,05	0,17	0,09	0,30	0,42
25	0,62	0,46	0,47	0,87	1,69	0,21	0,42	0,17	0,16	0,10	0,24	0,34
26	0,55	0,55	0,55	0,53	0,93	0,23	0,39	0,17	0,16	0,09	0,20	0,29
27	15,83	0,55	0,29	4,95	0,76	0,20	0,28	0,32	0,14	0,12	0,19	0,36
28	2,22	0,42	0,27	0,79	0,68	0,28	0,27	0,25	0,15	0,10	0,25	0,33
29	1,23	0,05	0,27	0,96	0,64	0,20	0,44	1,23	0,14	0,12	0,37	0,32
30	0,87		0,23	0,68	0,87	0,18	0,27	0,19	0,14	0,66	0,25	0,23
31	0,62		0,22		0,62		0,20	0,17		0,42		0,42

Source: Result of Analysis

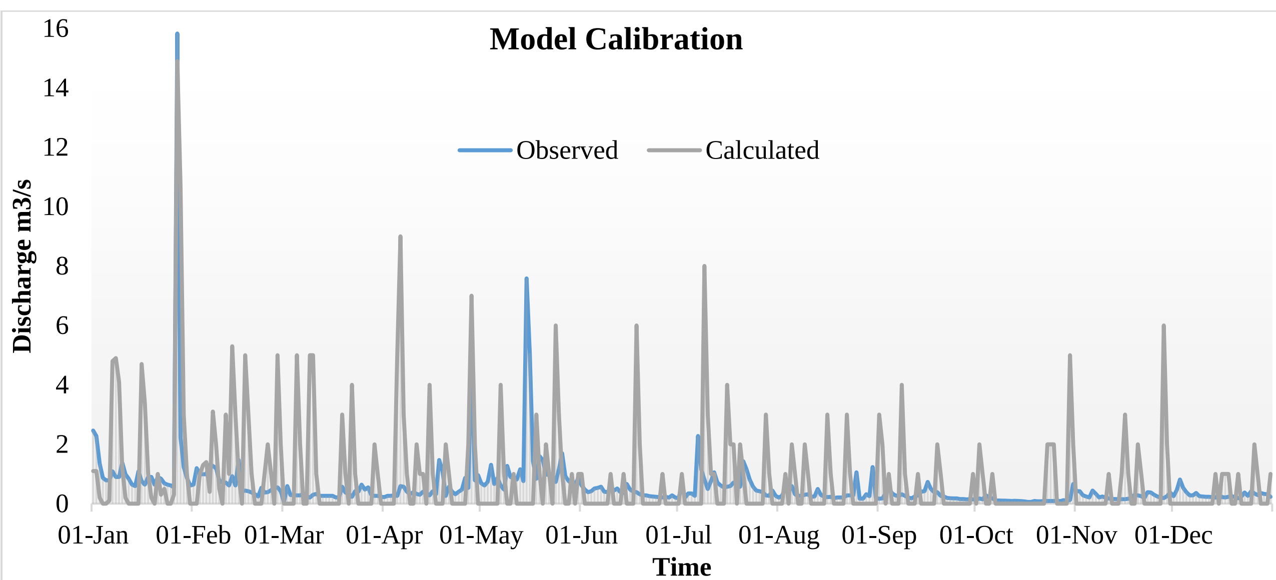

Fig. 3 Measured Discharge and Result of Calibration Computation

The subsequent analysis is the discharge analysis of flood. Therefore, the calibration will be subjected to peak discharge. The calibration will result in an approximately identical between the calculated and measured peak discharge. It is now concluded that the parameters for analysis are suitable for the river basin of Binjeita.

Table 12. Calibrated Parameters

CN	85	
Lag Time	60	min
Initial discharge	0.5	m^3/sec
Recession constant	0,1	
Ratio to peak	0,3	

Source: Result of Analysis

3.11. Analysis of Design of Flood with HEC-HMS

The flood design for several return periods is calculated by entering the design rainfall and calibrating parameters to HEC-HMS.

Table 13. Design of Flood

Return Period (Year)	Design of Flood (m^3/sec)
25	2.0
50	2.5

Source: Result of Analysis

3.12. Analysis of Water Level with HEC-RAS

This study used Steady flow surface profiles in the analysis. Based on available hydraulic data, the hydraulic model was based on a one-dimensional steady-state computation. HEC-RAS can calculate a one-dimensional water surface profile for steady, gradually varied flow in natural or constructed channels. The model also can be used in the subcritical, supercritical, and mixed flow regimes. The equation for basic profile calculation involves cross-section subdivision for conveyance calculations, Manning's n for the main channel, velocity weighting coefficient alpha, and friction loss.

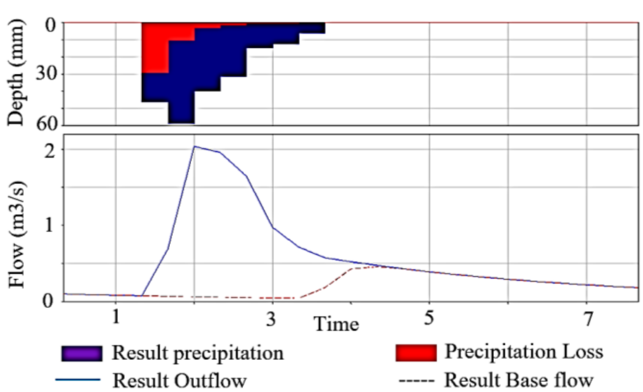

Fig. 4 Hydrograph of Peak Discharge at Location of Bridge 1 on Return Period of 25 Years

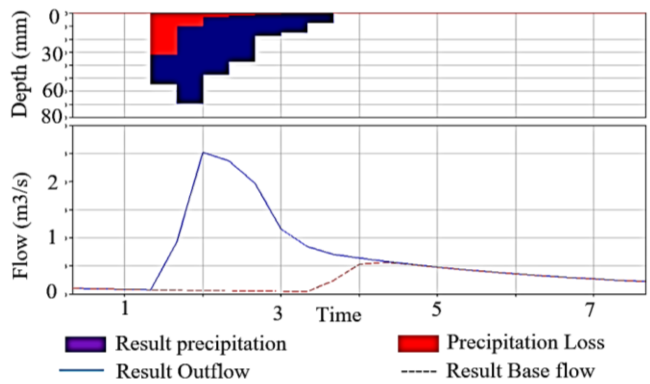

Fig. 5 Hydrograph of Peak Discharge at Location of Bridge 1 on Return Period of 50 Years

3.13. Cross Section of Binjeita River

The cross-section of the Binjeita river is obtained by field measurement of 32 cross-sections. The length of the river for analysis matches the drawing detail of the river from topographic measurement, which is 694.6 m long. The detail of the cross-section of the Binjeita river based on the topographic measurement is presented in Table 14.

3.14. Cross-Section Data for Analysis

Cross-section data for hydraulic analysis is the data from topographic measurement, as presented in Table 14. Below is a re-dimension of the cross-section from topographic measurement at the location of Bridge 1. The design cross-section at the location of Bridge 1 is shown in Fig. 6.

Table 14. Distance between Measured Cross Section on Location of Bridge 1 of Binjeita River

River Sta.	ID Sta	Length (m)	River Sta.	ID Sta	Length (m)
32	P1	25	16	P16	15
31	P2	25	15	P17	13.3
30	P3	23	14	P18	50
29	P4	25	13	P19	15.6
28	P5	25	12	P20	18,7
27	P6	25	11	P21	24.6
26	P7	7	10	P22	12.1
25	P8	25	9	P23	25
24	P9	25	8	P24	25
23	P10	25	7	P25	24.9
22	P11	17.9	6	P26	25
21	P12	25	5	P27	15.4
20	P13	25	4	P28	25
19	P14	12.5	3	P29	25
18	P14a (Bridge 1)	12.5	2	P30	32.1
17	P15	25	1	P31	0

Source: Result of Topographic Measurement

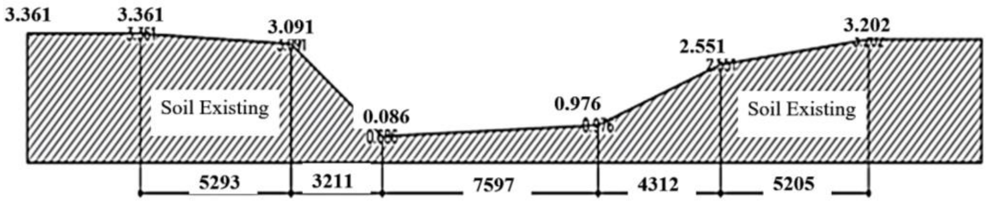

Fig. 6 Design Cross Section at Location of Bridge 1

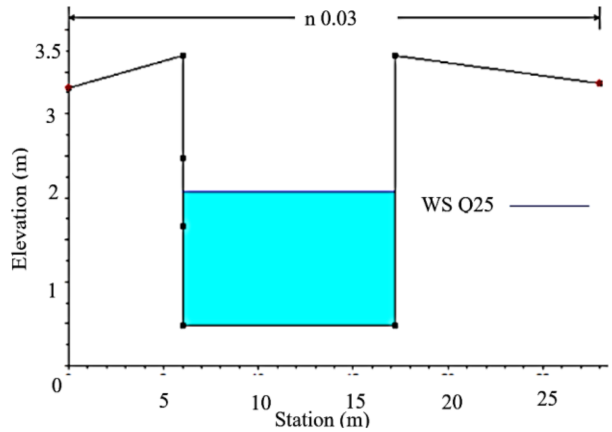

Fig. 7 Cross Section of P14a (Location of Bridge 1) at Return Period of 25 Years

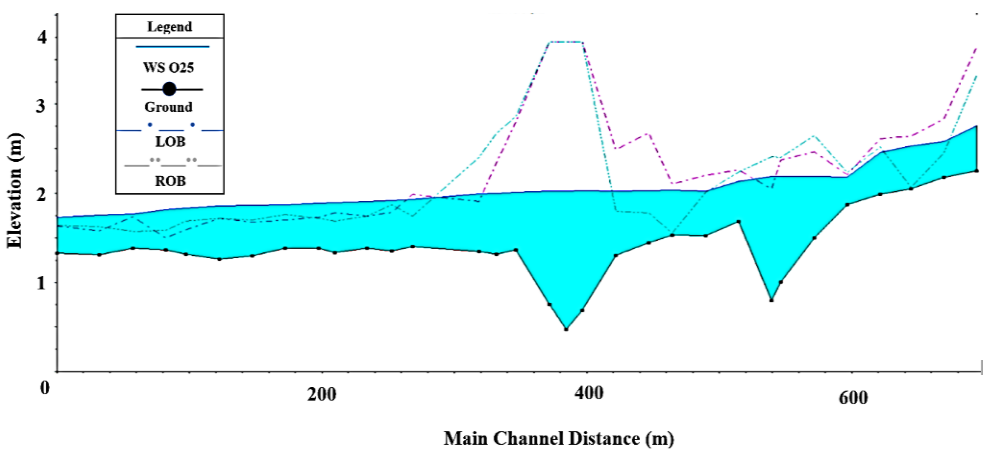

Fig. 8 Water Elevation Profile at Location of Bridge 1 during Design of Flood at Return Period of 25 Years

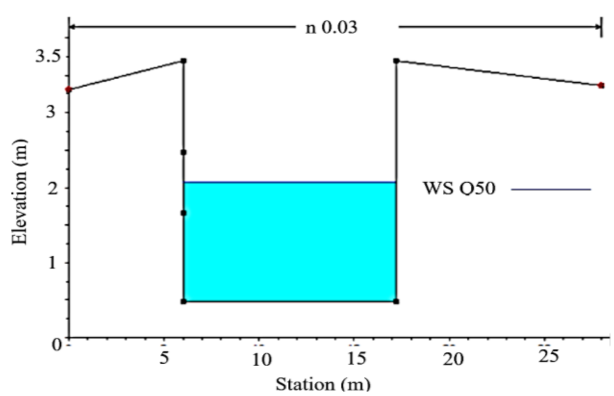

Fig. 9 Cross Section of P14a (Location of Bridge 1) at Return Period of 50 Years

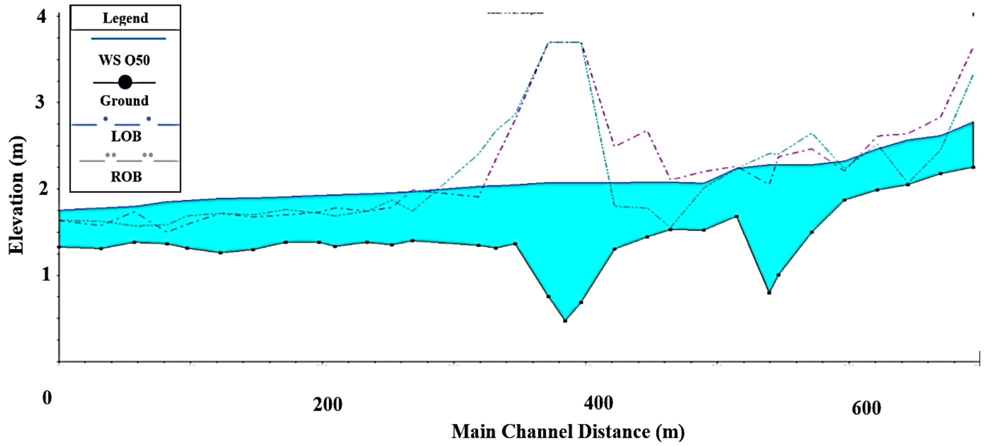

Fig. 10 Water Elevation Profile at Location of Bridge 1 during Design of Flood at Return Period of 50 Years

3.15. Analysis of River Capacity

The analysis of river capacity is conducted by utilising HEC-RAS 5.0.7. These parameters must be inserted before running HEC-RAS: cross-section and long section of the river, the riverbed's slope and the flood's design. An approach method where the design of flood meets the result of hydrology analysis is carried out to obtain the river capacity, with the boundary conditions downstream as the standard depth of the river and S = 0.001 as the average slope of the riverbed.

HEC-RAS 5.0.7 simulation with the design of flood at a return period of 25 years and 50 years on the location of Bridge 1 at Binjeita river gives results as shown in Fig. 7 to Fig. 10.

The total length of Binjeita river for flood analysis for the location of Bridge 1 is 694.6 m, and the Bridge is at a distance of 384.2 m from downstream. The analysis shows that several locations along the river have overflow potential.

4. Conclusion

The research concludes that the design of flood at a return period of 25 years and 50 years at the cross-section of Binjeita river for the location of Bridge 1 are 2 m3/sec and 2.5 m^3/sec, consecutively. The water elevation at the design of flood of both return periods does not cause overflow at the location of Bridge 1. However, overflow occurs in several areas upstream and downstream from Bridge 1, where its elevation varies to 50 cm high.

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