Re-Design of the Waru-Waru Weir Increased Emergencies Due to Increased Flood Discharges with Changes in Land Use

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Re-Design of The Waru-Waru Weir Overflow Due to Increased Flood Dishrage With Change Land Use

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Abstract

Waru-Waru Weir is one of the weirs in Bone Regency which was established in 2014, located in Batu Gading Village, Mare District, which serves 1000 Ha of agricultural land in Mare District, where the majority of the population work as rice farmers, and depend on the results of the rice fields for their living. In 2017 there was a flood which caused the upstream right retaining wall to collapse due to overtopping of the flow passing through the weir. This is because the capacity of the spillway is unable to pass the flood discharge estimated at Q100, so it is highly hoped that the function of the weir building will be optimized, so that it can support and maximize agricultural yields. The Waru-Waru Weir should be able to pass a discharge of Q100, but changes in land use upstream have caused a change in the value of Q100, so it is necessary to re-plan the spillway capacity of the Waru-Waru Weir. Based on the results of the hydrological analysis, it was found that Q100 was 778.18 m3/sec, so it was necessary to increase the width of the overflow weir by 20 m towards the right in height. The elevation of the additional overflow weir is readjusted to the highest rice field elevation and the width threshold at the intake gate so that +43.63 is obtained. Flood water level above the overflow weir at flood discharge Q100 = +46.88 so that there is still a 1.00 m guard height against the top elevation of the weir +47.88.

1 Introduction

Weirs are water buildings that function to elevated water levels, store water, stabilize water flow/irrigation, flood control and for diversion structure, where the planning and implementation of

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various supporting disciplines, such as hydrology, hydraulics, irrigation, river engineering, foundations, and environmental science to analyze environmental impacts in the construction of the weir. The Waru-Waru weir, which was built in 2014, has not been able to function optimally because in 2017 there was a flood that caused the retaining wall upstream to break because the flood water level had passed the peak elevation of the retaining wall which resulted in overtopping. This is because the spillway at the weir overflow is unable to pass the flood discharge at Q100 due to changes in flood discharge due to changes in land use upstream so that the value of the stream coefficient will be greater so that it is necessary to recalculate the flood discharge that passes through the weir overflow as an spillway and handling of the excess flood discharge. The Waru-Waru weir watershed consists of 2 watersheds, namely the Mare watershed and the Panilang watershed included in the Walanae Cenrana River Basin (WS).

In order for the planned spillway to pass the Q100 flood discharge, it is necessary to recalculate the hydrological analysis to determine the flood discharge under current conditions so that it can be planned again for the overflow at the Waru-Waru weir.

2 Research Method

The method used in this research is a survey method, where researchers carry out observations and surveys at the research location, namely at the weir location. The steps in the research process are as follows:

1. Literature Study
   Literature studies are conducted to gain knowledge and theoretical basis and methods that will be used.
2. Data Collection
   Collecting the data needed in this study, namely primary data collection taken from the research site and secondary data collection taken from related agencies. The data needed are:
   - Primary data obtained directly from the field, namely river width, river depth, and river bed elevation.
   - Maximum daily rainfall data for 10 years obtained from the ground station rainfall.
   - Watershed map made from Indonesia Topography Map scale 1: 50,000 to get the area of the watershed (DAS) catchment Waru-Waru weir
   - Waru-Waru river situation map obtained from field measurements.

3 Literature Review

3.1 Definition and Function of Weirs

Weir is a water building with fittings built across the river or sudetan which is deliberately made to elevated the water level or to get a high plunge, so that water can be tapped and flowed by gravity to the place that needs it. (Mawardi and Memed, 2002)

Weirs function, among others, to elevated the water level, so that river water can be tapped as needed and to control flow, sediment transport and river geometry so that water can be utilized safely, effectively, efficiently and optimally. (Mawardi and Memed, 2002)

3.2 Hydrological Analysis

A. Regional average rainfall

The amount of watershed rainfall (catchment rainfall) can be obtained by averaging point rainfall. The method used is the average method because there are only 2 rainfall posts around the watershed. Averages rainfall data from all stations in the watershed. (Harto, 2000). DAS rainfall is calculated using the following equation :

$$Hd = \frac{1}{N} \sum Hi \qquad (1)$$

where :

Hd = hujan DAS (mm)

Hi = hujan masing-masing stasiun (mm)

N = jumlah stasiun

B. Frequency Analysis and Probability Distribution

There are several parameters used in frequency analysis, namely as follows:

1) Average value

$$\overline{x} = \frac{1}{n} \Sigma_{i-1=x_i}^{n} \qquad (2)$$

where :

$\overline{x}$ = average rainfall (mm)

n = sum of data

x_i = precipitation at rainfall station to i

2) Standard deviation (S)

$$S = \sqrt{\frac{[\sum_{i=1}^{n}(x_i - \bar{x})^2]}{n-1}} \qquad (3)$$

S = Standard deviation

3) Variation of coefficient (Cv)

$$C_v = \frac{S}{\bar{X}} \qquad (4)$$

Cv = variation of coefficient

4) Skewness of coefficient (Cs)

$$Cs = \frac{n\sum_{i=1}^{n}(x_i-\bar{x})^3}{(n-1)(n-2)S^4} \quad (5)$$

Cs = skewness of coefficient

5) Koefisien *Kurtosis* (Ck)

$$Ck = \frac{n\sum_{i=1}^{n}(x_i-\bar{x})^4}{(n-1)(n-2)(n-3)(n-4)S^4} \quad (6)$$

Ck = Koefisien kurtosis

Some probability distributions Frequently used continuous:

1) Normal Distibution

$$X_T = \bar{X} + K_T S \quad (7)$$

X_T = An estimate of the value that is expected to occur with a return period T
$\bar{X}$ = Average value of variate
S = Standard deviation of variate values
K_T = Frequency factor, is a function of opportunity or period

2) Log Normal Distibution

$$Y_T = \bar{Y} + K_T S \quad (8)$$

Y_T = Estimated value that occurs with the return period T
$\bar{Y}$ = Average value of variate

3) Log Pearson Type III Distribution

$$X = \log X \quad (9)$$

$$Log\ \bar{X} = \frac{\sum_{i=1}^{n}\log x_i}{n-1} \quad (10)$$

$$S = \left[\frac{\sum_{i=1}^{n}(\log xi - \log x)^2}{n-1}\right]^{0.5} \quad (11)$$

$$Cs = \frac{n\sum_{i=1}^{n}(x_i-\bar{x})^3}{(n-1)(n-2)S^3} \quad (12)$$

$$Log\ X_T = \log \bar{X} + K.S \quad (13)$$

4) Gumbel Distribution

$$X = \bar{X} + S.K \quad (14)$$

$$K = \frac{Y_T - Y_n}{S_n} \quad (15)$$

Y_n = Reduced mean
S_n = Reduced standard deviation
Y_t = Reduced variate

$$Y_T = -\ln - \frac{T-1}{T} \quad (16)$$

C. Suitability test

1) Chi-Square test

$$X^2 = \sum_{i-1}^{n} \frac{\left(O_f - E_f\right)^2}{E_f} \quad (17)$$

X^2 = Chi-Square value are calculated
O_f = Number of observation values in the sub group to – i
E_f = The number of theoretical values in the sub group to – i
n = Number of sub group

The Chi-Square Test procedure is as follows :
a) Order of observation data
b) Group the data into *n* sub
c) Number of observation data Of each subgroup
d) Sum the data from the distribution equation used by Ef.
e) In each sub-group calculate the value :

$$(O_F - E_F)^2 \text{dan} \frac{(O_F - E_F)^2}{E_f} \quad (18)$$

f) Sum of all n subgroup values $\frac{(O_F - E_F)^2}{E_f}$ to determine the calculated Chi-Square value.
g) Determine the degrees of freedom Dk = G – R – 1 (nilai R = 2 for normal and binominal distributions)
The interpretation of the test results is as follows :
a) If the probability is more than 5%, the distribution equation is acceptable.
b) If the probability is less than 1%, the distribution equation cannot be accepted.
c) If the probability is between 1-5%, it is not possible to make a decision.

2) Smirnov Kolmogorov test

is often called a nonparametric fit test, because the test does not use a specific distribution function. The implementation procedure is as follows:

1. Sort the data (from large to small or otherwise) and determine the probability of each.
2. Sort the values of each theoretical probability from the data depiction results (distribution equation)
3. From the two odds values, determine the largest difference between the observed odds and the theoretical odds.
4. Based on the table of critical values (Smirnov-Kolmogorov test) determine the price of Do from the table below:

Table 3. Critical values Do for Smirnov-Kolmogorov test

N	Level of trust			
	0.2	0.1	0.05	0.01
5	0.45	0.51	0.56	0.67
10	0.32	0.37	0.41	0.49
15	0.27	0.3	0.34	0.4
20	0.23	0.26	0.29	0.36
25	0.21	0.24	0.27	0.32
30	0.19	0.22	0.23	0.29
35	0.18	0.2	0.21	0.27
40	0.17	0.19	0.2	0.25
45	0.16	0.18	0.19	0.24
50	0.15	0.17	1.36	0.23
N>50	1.07	1.22	1.36	1.63

Sources : Suripin, 2004

D. Effective rainfall

Effective rainfall is obtained by multiplying the design rainfall by the conveyance coefficient (Sosrodarsono, 1993). The general equation used is as follows:

$$Reff = Rt - d . Rt \quad (19)$$

$$\text{If, } 1 - d = C \quad (20)$$

$$\text{so, } Reff = Rt . C \quad (21)$$

where :

Reff = high effective rainfall (mm)

C = flow coefficient (Tabel 4)

Rt = high design rainfall (mm)

Table 4. Flow coefficient

Ground Cover	flow coefficient (C)	
	Low	High
Lawns	0.05	0.35
Forest	0.05	0.25
Cultivated land	0.08	0.41
Meadow	0.10	0.50

Parks, cemeteries	0.10	0.25
Unimproved area	0.10	0.30
Pasture	0.12	0.62
Residential areas	0.30	0.75
Business areas	0.50	0.95
Industrial areas	0.50	0.90
Streets		
Bricks	0.70	0.85
Asphalt	0.70	0.95
Concrete	0.70	0.95
Roofs	0.75	0.95

E. Regional rainfall distribution

In Indonesia, the rainfall interval is usually between 5-7 hours (Soemarto, 1986). The distribution of the design rainfall pattern is carried out in the following stages:

1. Calculate average rainfall up to hour t 2/3

$$Rt = Ro.(T/t) \tag{22}$$

$$Ro = R24 / T \tag{23}$$

Sehingga,

$$Rt = Rt24/T \times (T/t)2/3b \tag{24}$$

2. Calculating the average rainfall at hour t

$$Rt' = txRt-(t-1) \times R(t - 1) \tag{25}$$

where:

Rt : Average rainfall up to hour T (mm)

T : Time of day rain concentration

t : hours period to T

R24 : Effective daily rainfall height (mm)

Rt' : Rainfall height at hour T

Ro : Average daily rainfall

F. Hidrograf banjir

Teori dari hidrograf satuan sintetik Nakayasu dapat dirumuskan sebagai berikut (Soemarto, 1999):

$Qp = 3{,}6(0{,}3 \times Tp\ T\ )\ A \times Ro + 0{,}3$ (26)

where :

Qp = Peak flood discharge (m3/dt)

A = flow area (km2)

Ro = Rainfall height unit (mm)

Tp = rainfall start time to peak discharge flood (hours)

T0,3 = Time required by peak discharge reduction to 30% of peak discharge (hours)

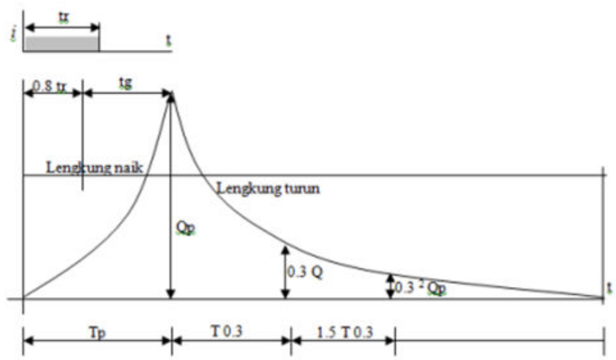

Figure 1. Hydrograph Sintetic Nakayasu (*Sources: Soemarto, 1999*)

The rain distribution pattern is determined by the rain concentration time :

$Tp = Tg+0.8Tr$ (27)

where:

Tp = Voltage time from the onset of rain to the peak of flood

Tg = The rainfall concentration time (h) depends on the length of the river (L),

$L< 15$ km $Tg = 0.4+0.058.L$ (28)

$L>15$ km $Tg = 0.21.L0.7$ (29)

Tr = Time unit of rainfall that costs 0.5

$Tg\ T0{,}3 = \alpha.Tg$ (30)

α = Coefficient of magnitude between 1.5 – 3

With the unit hydrograph calculated, the plan flood hydrograph for a certain return period can be calculated using the formula in the following table.

Table 4. Watershed hydrograph flood

U	R1	R2	R3	ΣR
U1	U1.R1	-	-	U1.R1
U2	U2.R1	U2.R1	-	U2.R1+U1.R2
U3	U3.R1	U3.R1	U1.R3	U3.R1+U2.R2+U1.R3
U4	U4.R1	U4.R1	U2.R3	U4.R1+U3.R2+U2.R3
U5	U5.R1	U5.R1	U3.R3	U5.R1+U4.R2+U3.R3

Sources : Soemarto, 1999

where:
U = unit of hidrograph (m3 /dt)
R = efective rainfall (mm)
Q = design flood hidrograf (m3 /dt)

4 Results and Discussion

4.1 Watershed Boundary Determination

Before determining the watershed (DAS), first determine the location of the planned location of the weir. From this location upstream, the watershed boundary is then determined by drawing an imaginary line connecting the points that have the highest contours to the left and right of the river under review (Soemarto, 1999). Figure 4.1 shows the Waru-Waru weir watershed at the control point at the upstream boundary of the study river span has an area of 128.18 km2 consisting of 2 watersheds namely Panilong watershed and Mare watershed.

Figure 1. Waru-Waru Weir Watershed

4.2 Rainfail data average

There are 2 (two) rainfall post stations around the Waru-Waru weir watershed, namely:

Table 5. Rainfall post station around the Waru-Waru weir

Num	Station	Latitude	Longitude	Data Type	Serial Data
1	Sta Hujan Mare	4°46'43.50"	120°16'6.20"	Manual	2008 – 2020
2	Sta Hujan Tellu Boccoe	4°49'60.00"	120°19'0.00"	Manual	2008 – 2020

The maximum rainfall from each rainfall post station is:

Table 5. Maximum daily rainfall of Mare station

Years	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Max
2008	31	39	33	31	33	26	30	17	17	22	22	24	39
2009	126	162	192	208	143	177	159	67	58	11	99	112	208
2010	26	27	25	31	33	30	35	34	34	30	34	33	35
2011	77	23	29	33	25	33	29	22	33	22	73	45	77
2012	44	29	32	35	27	33	35	21	15	27	29	29	44
2013	31	27	27	37	35	37	33	31	25	12	31	25	37
2014	27	17	25	31	35	33	22	33	0	5	15	22	35
2015	70	84	39	55	61	55	33	60	0	0	33	45	84
2016	37	52	52	51	58	75	61	54	49	75	24	63	75
2017	46	73	44	27	95	76	97	90	27	35	41	31	97
2018	65	32	112	35	41	87	55	28	56	13	63	56	112
2019	62	31	49	45	67	206	27	0	2	15	67	27	206
2020	29	38	68	53	77	57	53	7	22	41	67	19	77

Table 6. Maximum daily rainfall of Tellu Boccoe station

Years	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Max
2008	22	25	73	119	126	49	75	61	19	36	45	21	126
2009	29	19	43	108	25	50	62	0	0	11	67	27	108
2010	9	34	17	79	149	100	52	49	66	74	31	24	149
2011	66	0	31	68	68	14	81	0	19	38	34	75	81
2012	32	13	50	80	43	160	82	32	26	42	21	24	160
2013	55	35	21	80	92	58	91	47	55	22	38	56	92
2014	19	29	29	28	55	106	64	53	10	0	80	54	106
2015	23	28	19	56	50	89	10	0	0	1	24	27	89
2016	34	37	39	94	102	85	55	62	86	108	42	64	108
2017	21	85	110	42	301	236	300	65	42	110	44	21	301
2018	110	87	21	66	189	188	86	21	24	107	146	64	189
2019	42	79	105	171	194	325	63	20	20	0	20	84	325
2020													

From the data above, maximum daily rainfall data was then created using the average method because there are only 2 (two) rainfall post stations.

Table 7. Maksmum Rainfall Daily

No	Year	Maks Rainfall Daily		Avarege
		Mare	Tellu Boccoe	
1	2008	39	126	82.50
2	2009	208	108	158.00
3	2010	35	149	92.00
4	2011	77	81	79.00
5	2012	44	160	102.00
6	2013	37	92	64.50
7	2014	35	106	70.50

8	2015	84	89	86.50
9	2016	75	108	91.50
10	2017	97	301	199.00
11	2018	112	189	150.50
12	2019	206	325	265.50
13	2020	77	136	106.43

4.3 Design Rainfall

The results of the design rainfall calculation can be seen in the following table:

Table 8. Calculation of rainfall using the gumbel method

Return Period	Yt	K Factor	Xt
2	0.3665	-0.141	110.77
5	1.4999	0.996	177.72
10	2.2504	1.748	222.04
20	2.9702	2.470	264.56
25	3.1985	2.699	278.04
50	3.9019	3.405	319.59
100	4.6001	4.105	360.82
200	5.2958	4.803	401.91
500	6.2136	5.723	456.12
1000	6.9073	6.419	497.09

Table 9. Calculation of rainfall using the log person type III method

Return Period	G	Log Xt	Xt
2	-0.1566	2.007	101.67
5	0.7734	2.179	150.87
10	1.3490	2.285	192.61
20	1.8095	2.370	234.18
25	2.0398	2.412	258.22
50	2.5274	2.502	317.58
100	2.9933	2.588	387.00
200	3.4392	2.670	467.61
500	3.8154	2.739	548.56
1000	4.4426	2.855	715.81

After examining the distribution suitability test using the Smirnov-Kolmogorov suitability test and the Chi Square test, the results were obtained that the Log Person Type III hypothesis was acceptable.

4.4 Flood discharge and flood hydrograph

One of the parameters needed to calculate flood discharge is the flow coefficient (C) so it needs to be identified on the land use map, based on the identification results obtained:

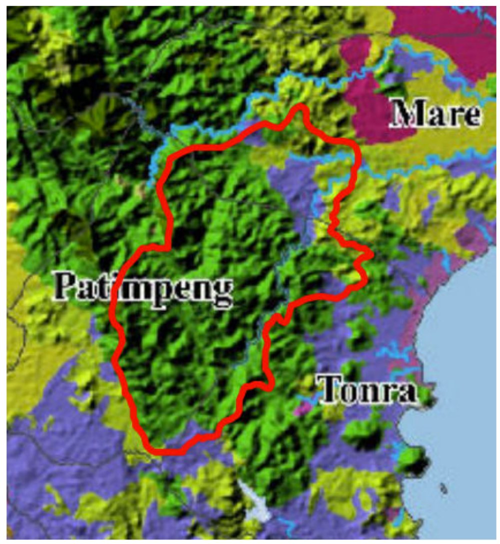

Figure 2. land use map of the Waru-Waru Bendung watershed

Table 10. The calculation of the flow coefficient can be seen in the following table:

Land use type	Surface area	Flow coefficient	Persentase	Sum of Flow coefficient
Land Agriculture	6.41	0.64	5	0.03
Ricefield	19.23	0.60	15	0.09
Forest	102.54	0.70	80	0.56
Total	**128.18**		**100**	**0.68**

From the results of these calculations, a drainage coefficient (C) of 0.68 is obtained which will be used in the Nakayasu formula to calculate flood discharge:

$$Qp = \frac{1}{3.6} \cdot \left(\frac{C.A.Re}{0.3\,.\,Tp + T_{0.3}} \right)$$

$Tg = 0.4 + 0.058\ .l$ (river length)
$Tr = 0.5\ Tg$
$Tp = Tg + 0.8\ Tr$
$T_{0.3}{=}\alpha\ .Tg$
Where :

Qp = peak flood discharge
C = flow coefficient
A = Watershed area (km2)
Re = effective Rainfall (mm)
Tp = time of start of flood to peak of flood hydrograph (hours)
T0.3 = time from peak flood to 0.3 times peak flood discharge (hours)
tg = concentration time
Tr = unit of time for rainfall (hours)
α = watershed characteristic coefficient
L = length of river (km)

The results of the flood hydrograph calculation using the formula above obtained the following results:

Table 11. Hydrograph Design flood in hours

Time	Discharge	Design Rainfall (mm/hours)					Total
	q	1	2	3	4	5	Discharge
(hours)	m^3/dt	137.57	35.76	25.08	19.97	16.86	m^3/dt
0	0.00	0.00					0
1	0.54	73.98	0.00				73.98
2	2.84	390.45	19.23	0.00			409.68
3	4.82	663.21	101.49	13.49	0.00		778.18
4	3.47	476.92	172.38	71.19	10.74	0.00	731.23
5	2.49	342.96	123.96	120.92	56.67	9.07	653.58
6	1.79	246.63	89.14	86.96	96.27	47.86	574.78
7	1.40	192.02	64.10	62.53	69.23	81.29	518.10
8	1.12	154.13	49.91	44.97	49.78	58.46	465.71
9	0.90	123.71	40.06	35.01	35.80	42.04	391.26
10	0.66	90.17	32.16	28.10	27.87	30.23	290.97
11	0.56	76.47	23.44	22.56	22.37	23.54	227.65
12	0.47	64.85	19.88	16.44	17.96	18.89	182.21
13	0.40	54.99	16.85	13.94	13.09	15.16	148.95
14	0.34	46.63	14.29	11.82	11.10	11.05	122.92
15	0.29	39.54	12.12	10.03	9.41	9.37	101.99
16	0.24	33.53	10.28	8.50	7.98	7.95	85.08
17	0.21	28.44	8.72	7.21	6.77	6.74	72.14
18	0.18	24.11	7.39	6.11	5.74	5.72	61.18
19	0.15	20.45	6.27	5.18	4.87	4.85	51.88
20	0.13	17.34	5.32	4.40	4.13	4.11	43.99

21	0.11	14.70	4.51	3.73	3.50	3.49	37.31
22	0.09	12.47	3.82	3.16	2.97	2.96	31.64
23	0.08	10.57	3.24	2.68	2.52	2.51	26.83
24	0.07	8.97	2.75	2.27	2.13	2.13	22.75
		0.00	2.33	1.93	1.81	1.80	11.69
			0.00	1.63	1.53	1.53	7.94

For the design flood peak discharge, it is obtained for Q100 = 776.18 m3/sec with the planned flood hydrograph graph which can be seen in the picture below:

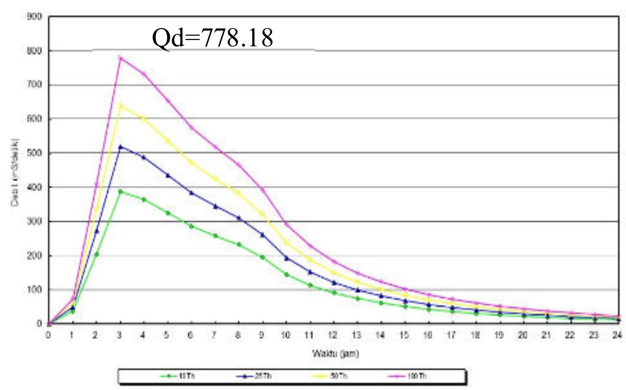

Figure 3. Nakayasu method flood hydrograph chart

4.5 Hydraulics Analysis of Weir

Based on the results of the hydraulics analysis, it is obtained that the capacity of the existing overflow weir is only able to pass a discharge of 326.91 m3 / d so that to pass the discharge Q100 = 778.18 m3 / d requires a redesign with several alternatives, among others:

1. Raising the retaining wall upstream
2. Make a side spillway upstream of the weir
3. Adding the width of the overflow spillway

After various considerations finally chosen to add the width of the overflow spillway, the calculation of the capacity of the overflow spillway used formula:

Calculation of the capacity of the overflow spillway weir Waru-Waru can be determined by the formula:

- long sill width :

$$Q = cl\, x \frac{2}{3} x\; b\; x \sqrt{2g} x\; H1^{1.5} \quad cl = 1.03$$

- short sill width :

$$Q = cd\ x \frac{2}{3} x\ b\ x \sqrt{2g} x\ H1^{1.5} cd = 1.3$$

From this formula, the calculation result is obtained :

No	Discharge	b	h	w	Overflow level	Flood Water Level	Top Level	Des
1	778.18	38	3.56	-0.56	44.88	48.44	47.88	The width of the overflow should be
2	326.907	38	2	1.0	44.88	46.88	47.88	Existing overflow width
3	451.273	20	3.25	1.0	43.63	46.88	47.88	Extend overflow width

So that an additional 20 m long lighthouse is needed to pass a discharge of 778.18 m3 / sec and reduce the height of the additional lighthouse by returning to the initial elevation before raising the lighthouse so that the additional spillway capacity is 451.27 m3 / sec. After obtaining the dimensions of the additional lighthouse, the design of the additional overflow weir is made as presented in Figure 4 and Figure 5.

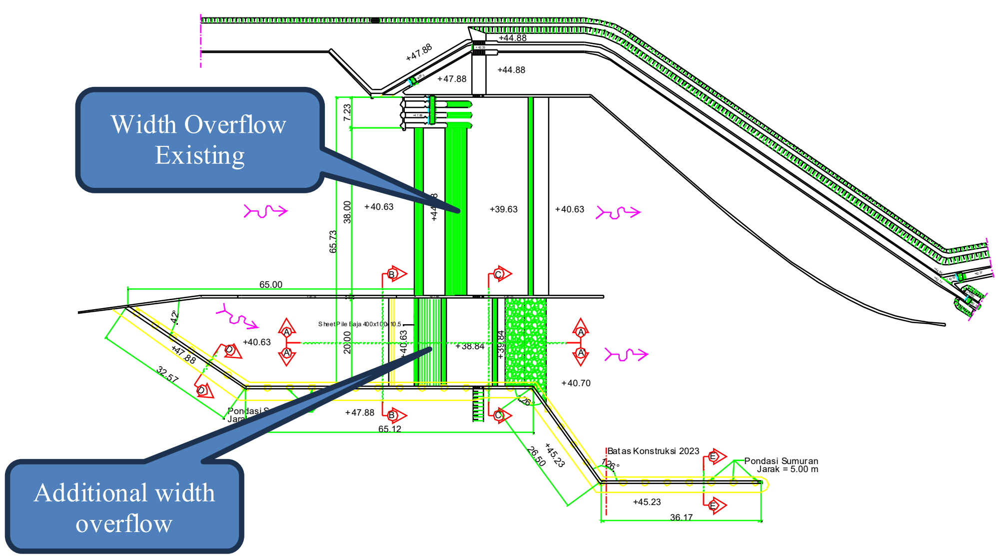

Figure 4. Lay out of Waru-Waru Weir

The elevation of the additional overflow weir is readjusted to the highest rice field elevation and the width threshold at the intake gate so that +43.63 is obtained. Flood water level above the overflow weir at flood discharge Q100 = +46.88 so that there is still a 1.00 m guard height against the top elevation of the weir +47.88

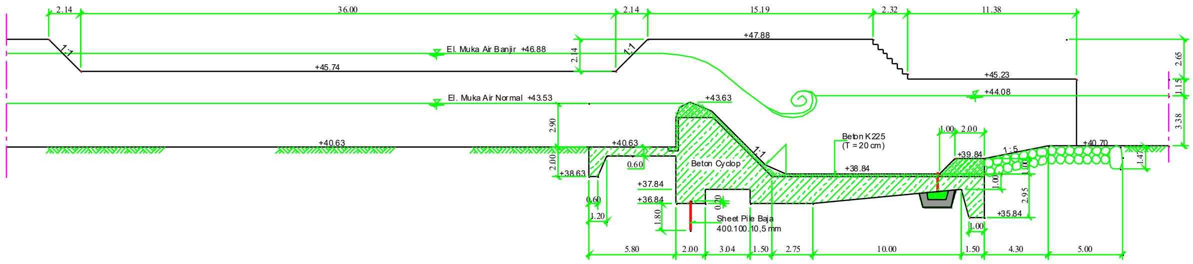

Figure 5. Longitudinal section of additional overflow of Waru-Waru Weir

Based on the results of the hydraulics analysis for that the additional overflow weir height is 2.90 m not the same as the existing overflow weir height of 4.25 m, this is due to maintaining the top elevation of the weir +47.88 m.

5 Conclusions

The results obtained from the re-design of the Waru-Waru weir overflow are:

a) The value of the conveyance coefficient (C) based on the results of identification on the use map for current conditions using the percentage method of each type of land cover obtained a value of C = 0.68
b) Based on the results of hydrological analysis obtained peak flood discharge at Q100 is 778.18 m3 / sec
c) An additional 20 m long overflow weir is required to the right of the existing overflow weir due to changes in flood discharge at Q100.
d) The flood water level at Q100 upstream is +46.88 with a freeboard of 1.00 m, then the top elevation of the weir is +47.88 which is in accordance with the existing top elevation of the weir
e) The height of the weir on the additional overflow weir is 2.90 m, there is a difference with the existing overflow weir of 4.25 m.

6 References

C. D. Soemarto, 1999, Engineering Hydrology, Erlangga Publisher, Jakarta.

Mawardi, Herman and Memed.2002.Hydraulic Design of Fixed Weirs for Irrigation, *Alfabeta,* Bandung

Hendra Hafid, 2021. Final Report Engineering Planning Wing Wall of Waru-Waru Weir. *Public Works and Spatial Planning Office of South Sulawesi Province.* Makassar

Sosrodarsono Suyono & Kensaku Takeda, 1993, Hydrologi for Irrigation, *PT. Pradnya Paramitha*, Jakarta

Soewarno, 1991, Hydrology Measurement and Processing of River Flow Data (Hydrometry), *Nova*, Bandung

Sri Harto Br. 2000. Hydrology: Theory, Problems, Solutions. *Nafiri Offset*. Yogyakarta

Suripin. 2004. Sustainable Urban Drainage System. *ANDI Offset*, Yogyakarta