Study of Progo River Morphology and Sedimentation of Intake Kamijoro Weir Bantul Regency, Special District of Yogyakarta

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Abstract

Irrigation Intake in Pijenan (2370 hectares) at Kamijoro weir Sungai Progo, Bantul Regency, is experiencing problems with sedimentation and blockages due to trash (stone and wood) so the intake does not function optimally.
BBWS Serayu Opak, Ministry of Public Works and Housing has made efforts by periodically dredging and constructing a Trashrack aimed at controlling sedimentation and waste in front of the intake.A comprehensive analysis through the morphological and hydraulic changes approach shows that the intake position tends to change the river channel pattern (meander). Besides that, the sediment transport rate of the Progo River is very large because it originates at Mount Merapi, which is still actively releasing eruptions and cold lava, besides that the rate is triggered by erosion of critical river watersheds lands (land erosion > 3mm/yr).For this reason, an alternative solution is needed to overcome sedimentation hydraulically, namely using the KRIB building on the side of the channel around the intake. In this study, analysis was carried out using HEC-RAS 6.1 software, both sedimentation and flow velocity.The simulation results of HE-CRAS 6.1 water level 2D and Sediment 1D software conditions before and after installing KRIB on the right side of the river, showed that after installing KRIB there was a reduction of 66.67% (bankfull condition) and 79.31% (100yr flood condition). Based on the Hjulstorm graph, the condition after the KRIB is installed shows that it is in the Transport in Suspension area, meaning the area where the sediment grains in front of the intake can be carried away by water velocity in bankful conditions. The simulation shows that there is a change in the morphology of the bottom and riverbanks around the intake which tends reduced sedimentation due to the deflection of the current towards the intake position so that the construction of KRIB can complete sedimentation at the intake in the long term.

Keywords: Kamijoro Weir, Progo River, Morphology, Intake, Sedimentation, KRIB Simulation.

1. Introduction

The Progo River upstream at Mount Sindoro and downstream into the Indonesian Ocean has a catchment area of 2438.33 km2 and the length of the main river is 143 km. The Progo River is a source of irrigation water and raw water by tapping water through the Kamijoro Weir Intake.
Kamijoro Weir is located on the Progo River with coordinates 7°52'42.51"S, 110°15'59.83"E**.**

Figure 1 Progo Catchment Area Location

Kamijoro Weir was completed in 2018, to irrigate the 2,370 hectare DI Pijenan. However, after a year of operation in 2019, the downstream Kamijoro irrigation area is still experiencing water shortages in Planting Season III. This is thought to have occurred due to sedimentation upstream of the intake and the blockage of the intake door due to trash (stone and wood).

(a) (b)

Figure 2 (a) Existing Intake Conditions (b) Sedimentation occurs upstream of the intake

BBWS Serayu Opak, the Ministry of Public Works and Housing has made efforts to carry out regular dredging, and the construction of a Trashrack aims to control sedimentation and garbage in front of the Intake, but it has not been maximized.
A comprehensive analysis using the morphological and hydraulic changes approach shows that the intake position tends to change the river channel pattern (meander) so that the side of the intake becomes an inner curve channel.
For this reason, an alternative solution is needed to overcome sedimentation hydraulically, namely using the KRIB building on the side of the channel around the intake. In this study, analysis was carried out using HEC-RAS 6.1 software, both sedimentation and flow velocity.

2. Solution Approach

2.1. Planned Rainfall Analysis

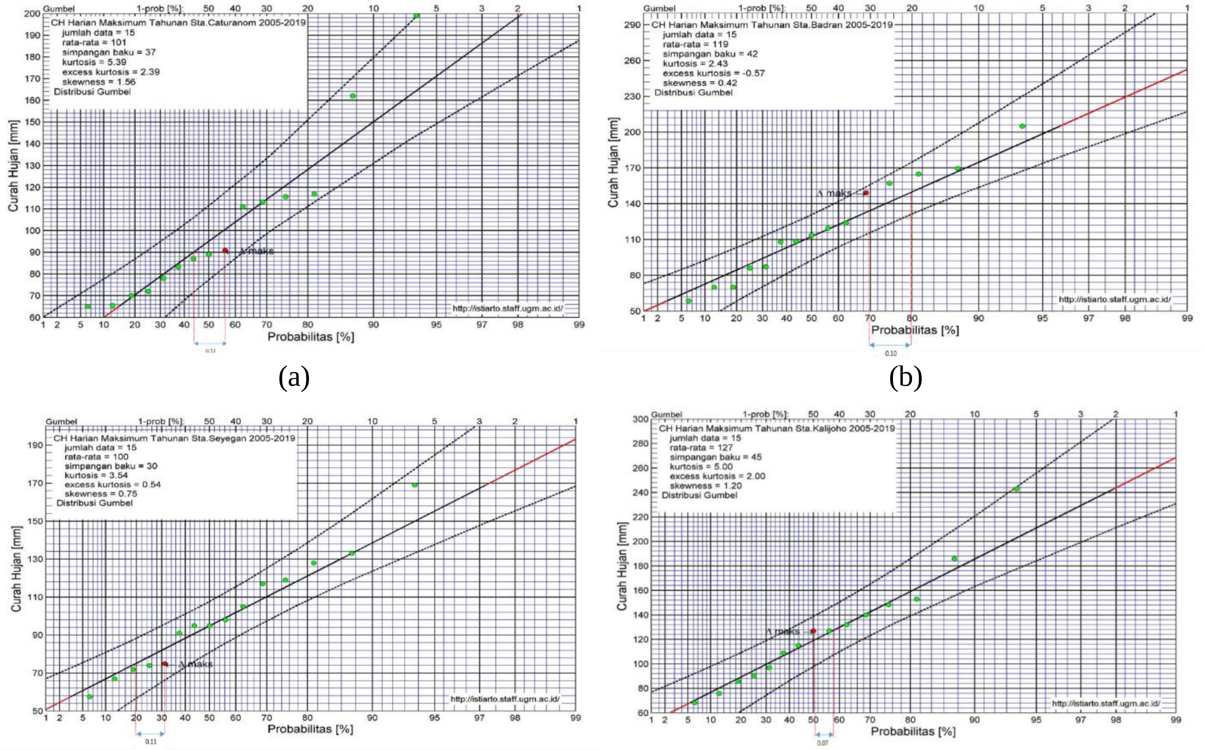
Used annual maximum rainfall data in the Kaliprogo watershed with the Kamijoro Weir as its outlet (downstream) which includes 4 (four) CH stations, namely Caturanom, Badran, Seyegan, and Kalijoho stations.

The rainfall data used is data for 15 years from 2005 to 2019. The maximum annual rainfall data can be seen in the table below.

Table 1 Annual Maximum Daily Rainfall in the Kaliprogo Watershed

Year	Sta. Caturanom	Sta. Badran	Sta. Seyegan	Sta. Kalijoho
	(mm)	(mm)	(mm)	(mm)
2005	87.00	86.00	71.80	148.40
2006	89.00	70.00	57.50	85.70
2007	162.00	119.70	91.00	90.40
2008	65.50	87.20	95.00	152.80
2009	300.00	108.00	67.00	68.30
2010	65.00	205.00	74.00	132.00
2011	70.00	149.30	75.00	109.00
2012	90.80	113.60	133.00	115.00
2013	111.00	157.30	95.00	140.00
2014	113.20	108.50	119.00	76.00
2015	116.90	70.00	117.00	127.00
2016	115.60	123.90	128.00	127.00
2017	72.10	165.00	169.00	343.00
2018	83.40	58.70	105.00	97.00
2019	78.00	169.50	98.00	186.00

Furthermore, the maximum rainfall data is tested for outliers on the Gumbel graph, shown as follows:



(a) (b)

(c) (d)

Figure 3 Station Outlier Test (a) Caturanom, (b) Badran, (c) Seyegan, (d) Kalijoho

From the results of the Gumbel graph plotting above, an Abnormality test was carried out with a Smirnov Kolmogorov critical value of 0.34 and 95% confidence Interval. Then we get max as follows:

Station	Δ max	grade	Remarks
Caturanom	0.12	< 0.34	Qualify
Badran	0.10	< 0.34	Qualify
Seyegan	0.11	< 0.34	Qualify
Kalijoho	0.07	< 0.34	Qualify

Figure 4 Results of Δmax for each station

Can be concluded that the value of max for each station meets the requirements (Δmax < critical).
Data validity was tested using the Double Mass Curve method by comparing the cumulative annual rainfall of each station to other stations (reference stations).
The QGIS software is used for area weight analysis (%) for each station using the Thiessen Polygon method.

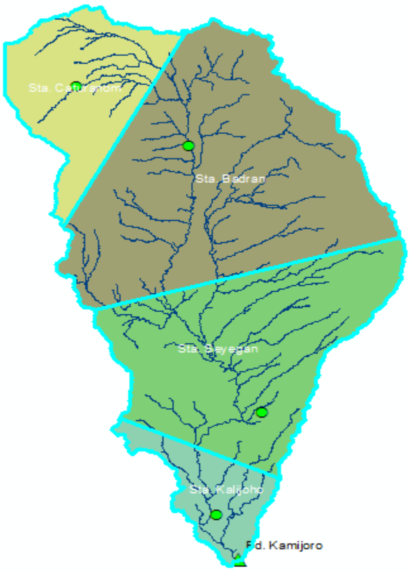


Figure 5 Results of Plotting Thiessen Polygon 4 Stations

After knowing the percentage area of influence of each rain station, the regional rainfall calculations for each station area.

From the results of the abnormality test using the Smirnov-Kolmogorov method, it can be concluded that the Gumbel distribution type meets the requirements for use in the calculation of the planned rainfall (Rt).

$$\mathbf{Rt = R + K_t . Sd}$$

Table 2 Gumbel Method Rainfall Distribution

T	Y	S	X_{TR}
2	0.37	24.60	116.8
2.33	0.58	24.60	122.0
5	1.50	24.60	144.7
10	2.25	24.60	163.2
25	3.20	24.60	186.5
50	3.90	24.60	203.8
100	4.60	24.60	221.0
200	5.30	24.60	238.1
1000	6.91	24.60	277.8

2.2. Analysis of Planned Flood Discharge

Calculation of the area of the reduction factor for the ARF area is carried out based on the SNI 2415:2016 standard.

In the calculation of the Synthetic Hydrograph Unit, the rain used is in the form of hourly rainfall distribution, so that the peak time of runoff can be known. Method Changes in monthly rain to rain with hourly distribution are done by using the PSA 007 cumulative rain distribution table with 12 hours of rain. Furthermore, the calculation of effective rain is accepted to obtain runoff from rain which has been reduced by Horton's method of soil infiltration. Calculation of flood discharge plan method ITB-2b.

Table 3 Calculation of Planned Flood Discharge method ITB-2b

Tr	ITB-2b Method
2	898.91
2.33	972.13
5	1290.27
10	1549.38
25	1878.45
50	2154.92
100	2429.34
200	2710.43
1000	3391.84

2.3. Sediment Transport Analysis Sediment

Sampling was carried out at 3 locations, namely: Bantar Bridge, Upstream Weir, and Downstream Weir. Suspended load sediment analysis (Qs) used 15 field data, then plotted onto a graph of the relationship between Qs and Qw, resulting in a regression value of $Qs=1.2983xQw^{1.9935}$.
Bed sediment load sediment (Qb) used the DuBoys equation (1979),

$$\tau = \gamma\ D\ S$$

Next, plotting was carried out to graph the relationship between Qb and Qw, resulting in a regression value of $Qb=961.44xQw^{0.7951}$.
Analysis of the total sediment discharge (Qt) was carried out by summing the values of Qs and Qb, plotting the Qt graph, resulting in a regression value of $Qt=896.83xQw^{0.8531}$, the graph shown as follows:

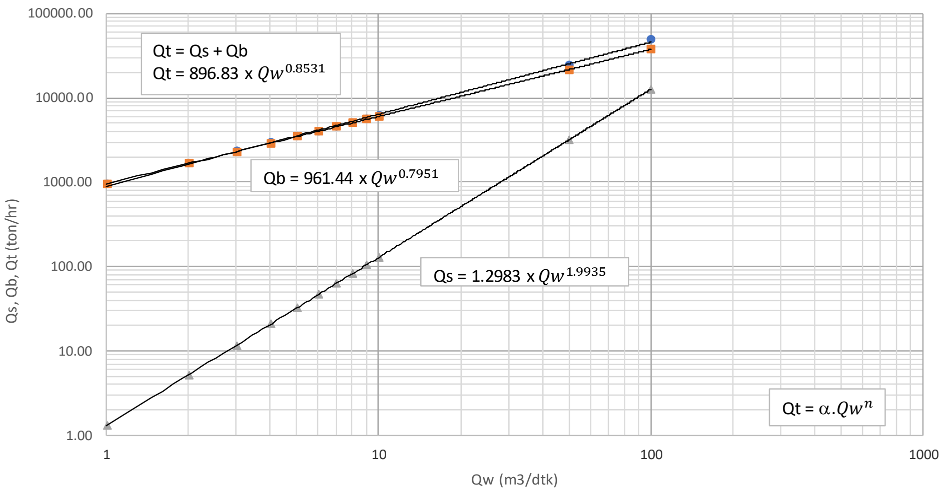


Figure 6 Regression Graphs Qs, Qb, and Qt

From the Qt equation, the total sediment transport can be calculated that the amount of sediment Qt = 10379.98 ppm, it can be interpreted that the river is experiencing heavy sedimentation.

2.4. River Geometric Analysis

2.4.1. Sinuosity Index (SI)

The data used to find SI (Sinuosity Index) are L and D values, using Google earth maps and topographic images. From the results of manual calculations, obtained: The length of the groove (L) of thalweg 332.12 m, (D) = 252.98 m, so the value of SI (L/D) = 1.31.

From the measurement results, according to Dury (1969), it can be concluded that the groove upstream of the weir is 1.31 < 1.5 (meander groove requirements), meaning that it does not meet the winding channel requirements.
Meanwhile, according to Rosgen, it can be concluded that the value of SI = 1.31 is included in the unstable flow pattern type (SI = 1.2 - 1.5).

2.4.2. Meander Stability
Calculation of the value of R (inner and outer) in the meander groove taken from google earth. The location being reviewed is in front of the Intake and is analyzed in Bankful condition.

Calculation of stability according to Ripley (1927), as follows:

Table 4 Comparison of results of manual calculations and according to Ripley

R Inner (visual) from google earth	R Outer (visual) from google earth	Width B	Depth d	Luas (A)	R meander growing 40√A	R meander Finish 110√A	Unit
167.96	334.22	90.28	1.26	204.71	572.30	1573.83	meter (m)
1538.71	2081.79	296.21	4.13	2203.43	1877.63	5163.49	feet (ft)

According to Ripley, the measured R value (visual, R inner=1538.71ft and R outer=2081.79ft) < R count (R developing = 877.63ft), so it can be concluded that the meandering process is still on going and has not yet reached stability. Furthermore, the meandering process will be completed when it reaches R = 5136.49ft.

2.4.3. Hydraulic Depth (y)
Calculation of hydraulic depth is carried out in bankful conditions (based on field data), using the equation J. Boussinesq (1877), as follows:

$$\boldsymbol{y = 6.35D\left(\sqrt{0.437 - \frac{x^2}{T^2}} - 0.433\right)\left(1 + \frac{xk}{ro}\right)}$$

Then it is plotted on to a graph of the relationship between distance (x) and depth (y).

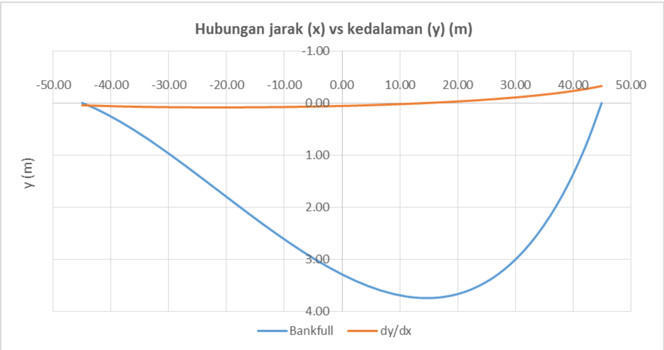


Figure 7 Correlation of distance (x) vs depth (y)

From the calculation results based on J. Boussinesq, it can be calculated that the hydraulic depth is obtained y=3.29m and the maximum depth is ymax=3.75m. Meanwhile, the existing depth of Bankful condition obtained from manual calculation is y=3.0m.
From the calculation results, it can be concluded that y count (3.75m) > y existing Bankful (3.0m), so that the depth upstream of the intake has not yet reached the hydraulic depth requirement for meander conditions.

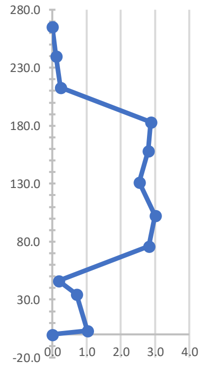
2.5. *HEC-RAS Modeling*

2.5.1. QMeasurable and Qflood Validation

Based on information from the gate guard of the Kamijoro Weir, the flood water level that had occurred in the Progo River on the section was about ± 70m from As Weir, namely at an elevation of +26m or as high as ±4 m from the bottom of the channel.

Next is the calculation of the measured Q river capacity which will be validated with the theoretical Q Synthetic Hydrograph Unit, presented in the following:

Table 5 Calculation of Measured Q Based on The Maximum Water Level of the Field

No.	Distance from edge (m)	Water Depth (m)	Area (A) (m²)	Total A (m²)	(P) (m)	R Hydraulic R = A/P (m)	Slope (S) (m/m)	Velocity (C=45) $V=C.R^{1/2}.S^{1/2}$ (m/s)	Q Measured (m3/s)	Drawing Sketch
1	0.000	0.000	0.000							
2	3.000	1.030	1.546							
3	34.700	0.700	27.431							
4	46.300	0.180	5.108							
5	76.300	2.820	45.010							
6	102.300	3.000	75.664	430.372	278.666	1.544	0.002	5.516	2374.00	
7	131.300	2.530	80.190							
8	158.200	2.800	71.698							
9	183.200	2.880	71.009							
10	213.250	0.230	46.738							
11	240.250	0.110	4.599							
12	265.250	0.000	1.379							

It can be concluded that the reservoir of the Progo River in the upstream section of the weir, the highest water level that has ever occurred, which is as high as 4m from the bottom of the intake can be achieved at a discharge condition of 2374 m3/s (Q measured). The amount of this discharge is equivalent to a 100-year return flood (2429.34 ^{m3}/s).

2.5.2. Validation of Manning's n value

Before performing calculations and simulations of HEC-RAS 6.1 using Q Bankfull and Q100yr, first validation of the Manning n value was carried out by finding the measured Q value of field results conditioned by Bankfull compared with Q2.33yr ITB-2 (HECRAS) method);

The measured Q field is conditioned Bankful at Sta.822 (as an example of a review), obtained a QBankfull value = 966.58 ^{m3}/s with d=3.0 m.

Table 6 Calculation of Measurable Qbankful (Sta. 822)

No. (m)	Distance from edge (m)	Water Depth (m²)	Area (m²)	(P) (m)	R Hydraulic R = A/P (m)	Slope (S) (m/m)	Velocity (C=45) $V=C.R^{1/2}.S^{1/2}$ (m/s)	Q Measured (m3/s)
0.000	0.000	0.000						
101.000	0.000	0.000						
113.828	2.5	15.971	04.706	167.365	1.223	0.002	4.722	966.58
122.900	3.0	24.857						
175.111	2.5	143.580						
191.284	0.0	20.297						

It can be concluded that the simulation results of Q2.33yr were chosen to approach Bankfull debit, carried out at Sta. Which is equal to the measured Q field.
The results of running Q2.33yr obtained a value close to the QBankfull field of Q2.33yr=966.78 m^3/s, and the Manning roughness value of n=0.031, which will be used for sediment simulation and design KRIB.

2.5.3. Sediment Transport Simulation
HE-CRAS simulation was carried out to determine the total amount of sediment transport that occurred upstream of the weir and in front of the intake.
The results of the analysis of sedimentation and scour heights at daily discharge using Bantar water forecast post data for a year are as follows:

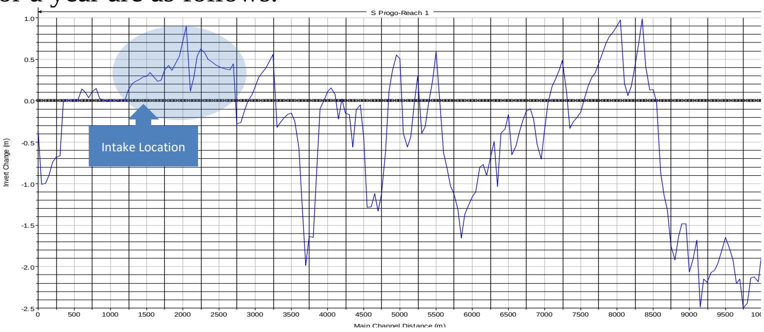


Figure 8 Sedimentation Height to Riverbed

From the analysis results, it can be concluded that at the location along the 1400 m from the upstream of the weir (points 1350-2750) and the upstream location of the Intake, the maximum sedimentation height is 0.89 meters.

2.5.4. Sedimentation Simulation with KRIB
Simulation of the effect of KRIB on sedimentation in QBankfull and Q100yr flood conditions was carried out.

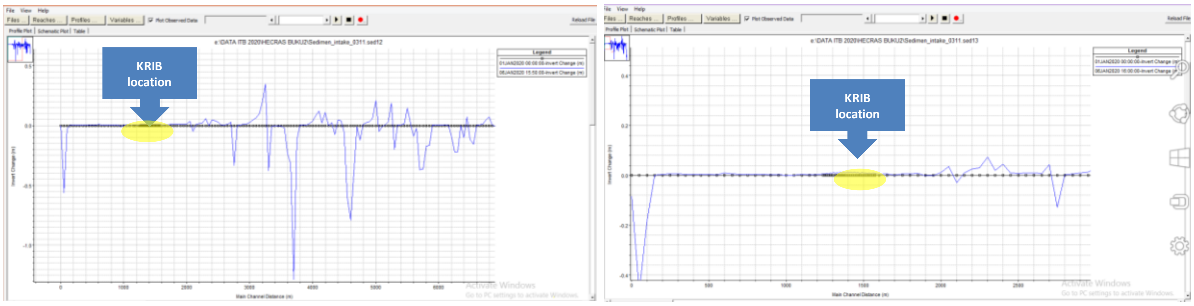


Figure 9 Simulation of Sedimentation after KRIB was installed QBankfull and Q100th Conditions

From the above running results, it can be concluded that in the KRIB building area, sedimentation and scour almost did not occur.

2.5.5. Simulation of flow velocity with KRIB
KRIB building is designed according to SNI standards, with the trial and error method, then the optimum design is obtained with the following conditions:
KRIB width 0.5m, KRIB length = 38m, distance between KRIB = 23m, and number of KRIB n = 15pcs;
The KRIB is installed perpendicular to the flow.
Simulation of 2D flow patterns before and after the KRIB building at QBankfull and Q100yr conditions, as follows;

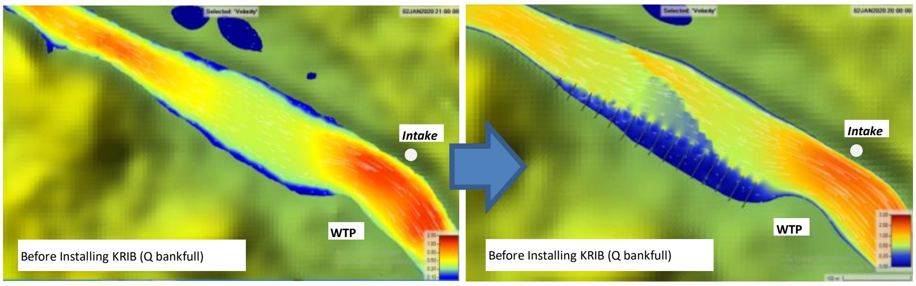


Figure 10 Simulation of Qbankfull Flow Pattern (Before and After KRIB)

In the QBankfull simulation there is a change in flow after the KRIB is installed, it can be seen in the change in the direction of the velocity vector and the color gradation of the velocity.

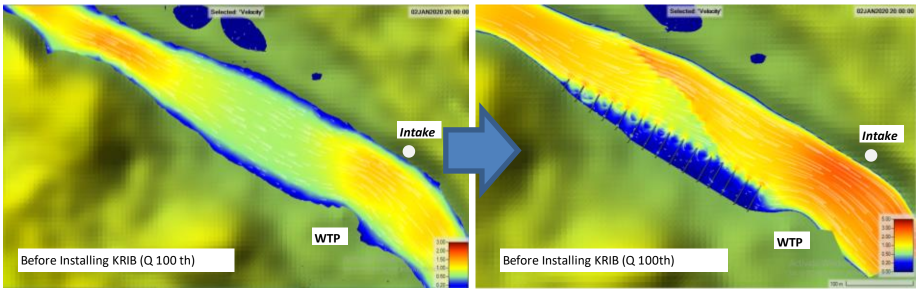


Figure 11 Simulation of Q100yr Flow Pattern (Before and After KRIB)

For Q100yr simulation, there is a change in flow after the KRIB is installed, it can be seen that there is a change in the direction of the velocity vector and the color gradation of the velocity. In addition, in the simulation after installing KRIB, the speed tends to be higher on the right side of the river (intake side). Velocity in cross-section before and after the KRIB building at QBankfull and Q100th conditions, as follows;

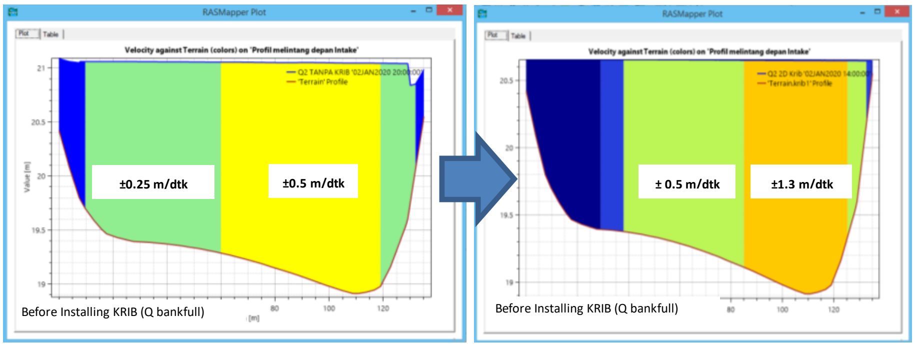


Figure 12 Distribution of QBankfull velocity Before and After Installing KRIB

For Qbankfull simulation an increase in speed after KRIB was installed on the Intake side, it can be seen in the yellow to orange color change (change from ±0.50 m/s to ±1.3m/s).

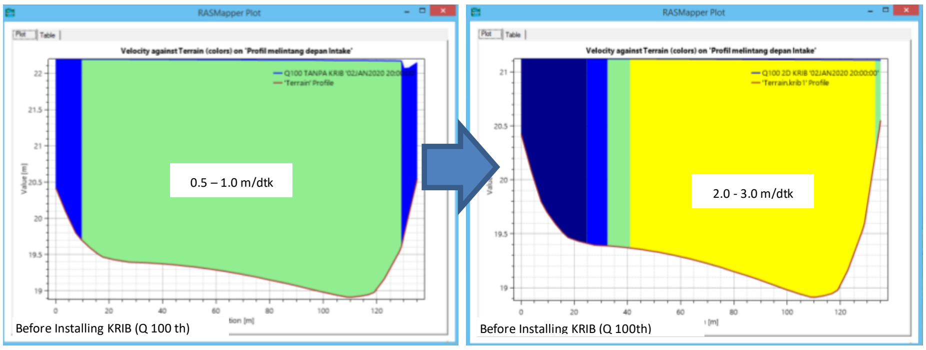


Figure 13 Distribution of Q100yr velocity Before and After Installing KRIB

For Q100yr simulation there is uniformity of speed after the KRIB is installed (green color changes to yellow). There is also an increase in speed at the Intake to v=2.0 to 3.5 m/s, due to the narrower cross-section of the river.

While the simulation of longitudinal cutting speed before and after the KRIB building in QBankfull and Q100yr conditions, is as follows:

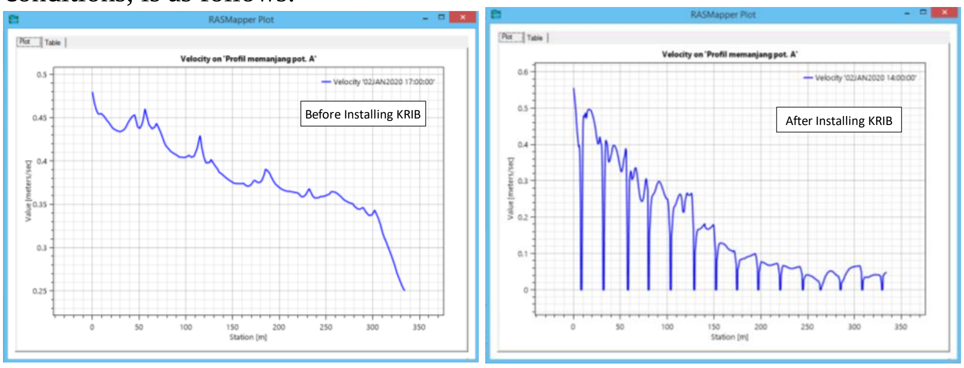


Figure 14 Longitudinal Cutting Speed Before and After Installing KRIB For Qbankfull

In the simulation of Qbankfull conditions, it is concluded that the flow velocity on the longitudinal section before KRIB (max=0.48m/s and min=0.25m/s) and after KRIB is (max=0.48m/s and min=0.03m/s). It can be concluded that there was a reduction in speed after KRIB was installed by 66.67%.

For the next, in the simulation of Q100yr conditions, the velocity in the longitudinal section before KRIB (max=0.77m/s and min=0.52m/s) and after KRIB is (max=0.77m/s and min= 0.06m/s). It can be concluded that there was a reduction in speed after KRIB was installed by 79.31%.

2.5.6. Correlation of Speed and Transport of Sediment Grains

The KRIB design calculation is based on the condition of the extent to which changes in velocity due to the KRIB building can flush sediment grains upstream of the intake. Calculations are carried out in Bankful conditions.

For example, the analysis is reviewed in longitudinal sections, shown as follows:

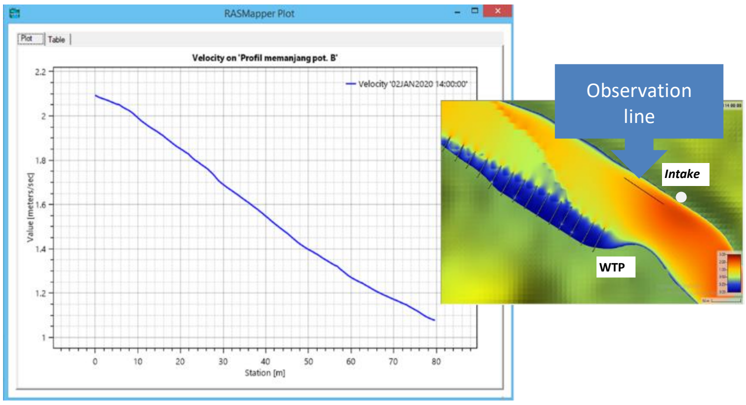


Figure 15 Graph of Flow in Line Observation

Velocity onvelocity in view area is (v=1.3m/s - 2.1m/s). Furthermore, the relationship between velocity and grain diameter (d50=0.13mm) is included in the Hjulstorm graph.
For example, the analysis is reviewed in longitudinal sections, shown as follows:

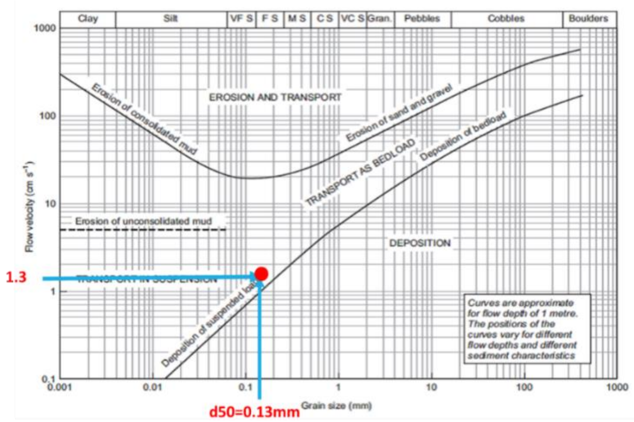


Figure 16 Plotting the Hjulstorm Graph.

The results of plotting, it can be concluded that the confluence of flow velocity and diameter of sediment grains indicates that they are in the Transport in Suspension area, meaning the area where sediment particles can be carried by water following the river flow.

3. Conclusions

From the results of the analysis and discussion, the following conclusions can be drawn that the morphology of the Progo river is still dynamic, due to the sediment source from cold lava/ lava from mountain eruption, based on the results of the grain analysis test.
Based on the reference of Dury (1969), it can be concluded that the type of groove pattern in the upstream weir is 1.31 < 1.5 (meander groove requirements), meaning that it does not meet the requirements of meander groove, while based on the Rosgen reference, it can be concluded that the results of the SI value analysis = 1.31 is included in the type of unstable flow pattern (SI = 1.2 - 1.5). Based on the equation of J. Boussinesq, it is concluded that the hydraulic depth (y=3.75m) at the upstream intake has not yet reached the requirements for the hydraulics of the existing condition/ bankfull (y=3.0m). According to Ripley, at the upstream curve of the intake, the measured R-value < R count, indicating that sedimentation will occur intensely in the inner curve and local scouring will occur, making the river bank unstable.

The geometric shape of the Progo River channel has a meandering channel, from the upstream (foot of the mountain peak) to the Kamijoro weir. The morphological condition of the Progo river affects the sediment transport rate in the heavy category Qt=10379.98 > 10000 ppm (10 kg/m3/s).
The results of the HEC-RAS 6.1 simulation before and after the installation of the KRIB on the right side of the river, showed that after the installation of the KRIB there was a reduction of 66.67% (bankfull condition) and 79.31% (100yr flood condition).
In the condition after the KRIB is installed, based on the Hjulstorm graph that connects the velocity v=(1.3-2.1m/s) with the grain diameter (d50=0.13mm), it shows that the point is in the *Transport in Suspension area*, meaning the area where the sediment grains are in front of the intake. can be carried away by the velocity of the water under bankful conditions.

4. References

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