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An Analysis of Canal Blocking Construction as a Water System Restoration on Peatlands in Sei Ahas Village Kapuas District Central Borneo

Eka Apriliasi[1], Prof. Dr. Ir. Mohammad Bisri, MS[2], Dr. Ir. Ussy Andawayanti, MS[2]
[1]Student of Magister Program of Water Resources Engineering of University of Brawijaya
[2]Lecturer, Department of Water Resources Engineering, Faculty of Engineering, University of Brawijaya
Mayjen Haryono Street 167 Malang 65145 – Phone. (0341) 567886
Email[1]: aprilliasi064 @ gmail.com

Abstract— *Central Borneo was a priority area o BRG, one of this BRG activity location was Sei Ahas Village, Mantangai Sub-District. Although, canal construction has been done previously for the management of water resources system. Forest fire and peat lands were still becoming the problems. Based on this condition, it needed to do a data analysis regarding to the construction of canal blocking in the area of Sei Ahas Village. To determine the capacity of flow profile in its channel, it required to a flow simulation with HEC-RAS program. From the process of hydraulic simulation, it indicated that the mean water level was able to get beyond the land elevation but not maximally, the average of inundation height was only 0,001 m or 1 mm. Thus, the canal blocking on primary canal was less maximal, while on secondary canal it appeared the decrease of water level on the downstream canal. The handling alternative to overcome this water debit was to fix the existing canal blocking and add two floodgates on each secondary canal and provide four units of water pump.*

Keywords— *Forest Fire, Peat Land, Canal Blocking, Water System, HEC-RAS Program.*

I. Introduction

The peat lands in Central Borneo have covered a fairly large area, in approximately 3,472 million Ha, or about 21,98% from the total area of Central Borneo that reaches to 15,798 million Ha. The main problem of peat ecosystem damage in the area of Central Borneo is the poor consolidation which aims to prioritize the sustainable policy of peat ecosystem peculiarity and focus on peat ecosystem peculiarity as valuable ecosystem but weak.

The Central Borneo is one of priority area of BRG which acquire a great concern, especially in Kapuas District. The region of Kapuas District which becomes the location of BRG activity is Sei Ahas Village, Mantangai Sub-District. The forest and peat land damage in Sei Ahas Village and Mantangai Sub-district are commonly caused by repeated fires. The land clearing and canal construction at that time have affected to a serious damage and deforestation of that area because of drainage and forest fire.

Regarding to those condition, this research aims to analyze the data of canal blocking construction in the area of peat lands in Central Borneo as a solution of peat land prevention and manage water resources system.

Research Problems

The following are problems which will be discussed in this research:

1. How is the analysis on existing water system and canal system to the construction of existing canal blocking in area of research location?
2. What are the appropriate handling strategies during water deficiency or excess on water system in area of research location?
3. How are alternative channel systems and technical specifications for peat wetting to overcome problems that occur in area of research location?

Research Limitations

Considering to the implementation plan of peat restoration system on this research is too wide, therefore, the researchers restrict the planning on these following points:

1. Firstly, this research will examine on the existing linkage system, drainage capacity, and canal blocking, then continue to the analysis and planning with a review on hydraulic aspect through hydrology analysis that relates to the canal blocking planning, but do not take into account the aspects of tidal hydrology.
2. This research was conducted comparing high tide level elevation without floodgates and with no floodgates, to analyze which zones are experiencing water shortages and which zones are experiencing excess water.
3. The rain catchment area is identified only on the area of which runoff water is possible to load the area of swampland restoration and not discuss water quality problems or sedimentation that occurs in primary channels.

Research Objectives

The following are objectives of this research:

1. To analyze the existing water system and infrastructure which adjust to eligibility rules technically especially in peat swampland of Central Borneo.
2. To analyze and find handling alternative during water deficiency or excess in location of research.
3. To determine the handling on water drainage system to overcome the problems in location of research that meet to the criteria for peat land dampening.

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II. Study Location

This research location was taken place at Sei Ahas Village, Mantangai Sub-district, Kapuas District, Central Borneo, Indonesia. Sei Ahas Village has an area of about 26.713 Ha or ± 267,13 Km^2 which covers to west and east of Kapuas River. The land location in Sei Ahas Village was an area of peat land that became one of peat restoration study in Central Borneo. Particularly, Sei Ahas was a lowland area with the surface height less than 11 m from sea level (A Field Survey, 2017). Moreover, most of the area were peat lands with depth level between 0,5-8 m. next, the hydrology in this village was influenced by the ebb and flow of Kapuas River, thus, the this area was prone to flood river overflow, especially during rainy season.

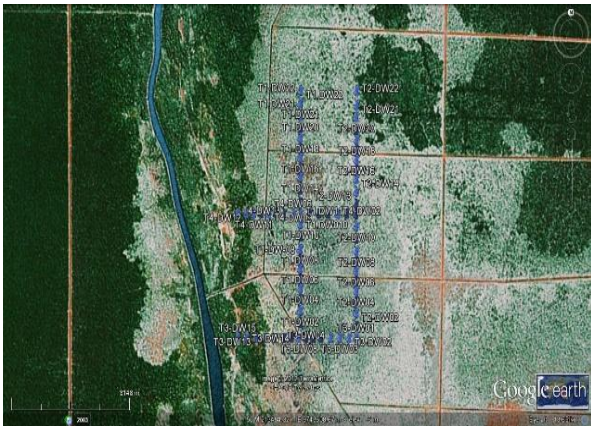

Fig. 1. Linkage Location Map of Sei Ahas (source: http://loketpeta.pu.go)

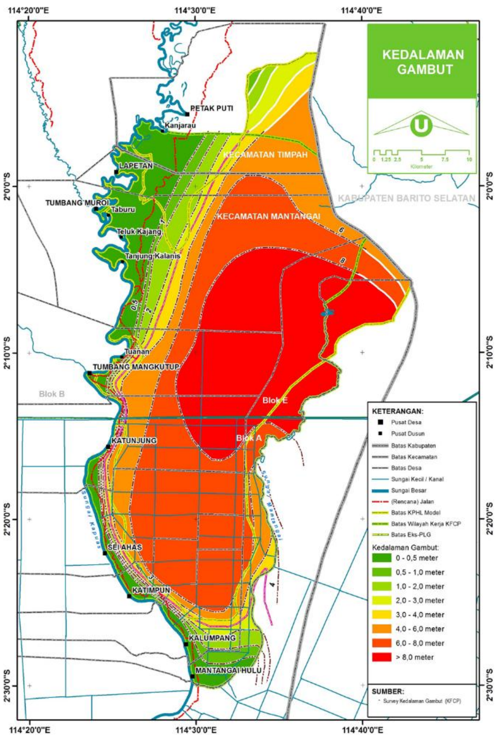

Fig. 2. Peat Depth Map for Central Kalimantan Province (source: http: //loketpeta.pu.go)

III. Data Collection

This research needed supporting data that were required to do calculation and analysis. The following were data needed for the activity of calculation and analysis on this research are the rainfall data of daily maximum and climatological data were acquired from rainfall station around the research location, in Mampai Sub-district, Kapuas District which was approximately ± 52.9 km from the research location and in Anjir Palembang Sub-district, Kapuas District which was approximately ± 57,7 km from the research location. The rainfall data was during 2010-2019.

The first stage of this research was to collect the secondary data which was regarded as systematic procedure that aimed to collect the data of research. The collected data was secondary data from hydrology and hydrometer condition of survey area. The next stage was to identify the research location based on the data which have been collected through calculation method manual hydrology analysis with assistance of Microsoft Excel software. The third stage, the data inputted into a canal model on HEC-RAS program were filling geometry of river data or flow discharge by inputting river Long Section and Cross Section. After that, the input of rainfall data which needed or other data that would be analyzed in this research. The debit would be inputted in canal design on HEC-RAS program which was then examined and resulted the repetition of canal debit on this research. Moreover, through HEC-RAS program, it was aimed to design long section profile of primary canal in high tide situation when there was no rain.

The flow diagram of research would be presented below on figure 3.

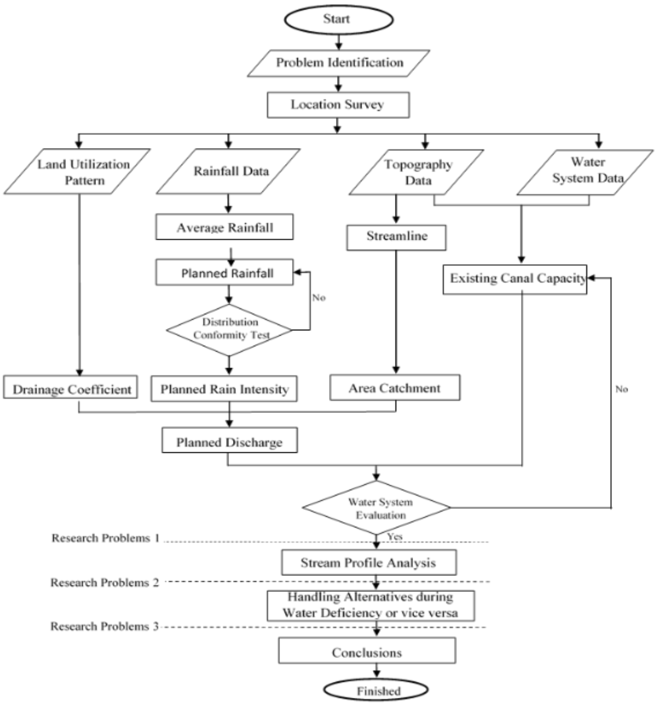

Fig. 3. Research Flow Diagram

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IV. DATA PROCESSING

Rainfall Data

On the research location, the logging activity of daily rainfall comprised of daily rainfall data with an observation period from 2009 to 2018 (10 years). Moreover, the research location also has the nearest post of rainfall logging in Mampai Sub-district, Kapuas District and Anjir Palembang Sub-district, Kapuas District.

Rainfall Data Analysis

Based on the calculation analysis from the rainfall data testing, it referred to the result that was listed in following recapitulation table (table 2).

Based on the recapitulation result of rainfall testing above, this research concluded that the RAPS consistency test for both rain station of Mampai and rain station of Anjir Palembang was equally approved. While, for the trend nonexistence test, both rain station did not have trend data, even the data variance of both stations were similarly homogenous and random. However, this research found a little difference on outlier test, which Anjir Palembang station resulted to the lower limit of outlier test was not approved.

TABLE 1. Annual Maximum Rainfall Data

Year	Meteorological Station of Mampai		Meteorological Station of Anjir Palembang	
	Maximum Rainfall (mm)	Annual Rainfall (mm)	Maximum Rainfall (mm)	Annual Rainfall (mm)
2010	314	821	413	3521
2011	421	1634	523	1971
2012	517	2171	438	1826
2013	456	2542	538	3149
2014	471	2124	550	3205
2015	310	1667	488	2196
2016	421	2796	398	2192
2017	389	2321	362	2545
2018	448	2352	528	2003
2019	402	1353	262	1059

(Source: Calculation Result)

TABLE 2. Recapitulation of Rainfall Data Testing

Station	Test					
	RAPS Consistency	Trend Nonexistence	Stationer		Spearman Persistence	Outlier
			F	t		
Mampai Station	OK	H0 is approved No identification of the trend	H0 is approved Data variance is stationary/homogenous	H0 is approved Data is stationary/homogenous	H0 is approved Data is random	Lower Limit is Approved Upper Limit is Approved
Anjir Palembang Station	OK	H0 is approved No identification of the trend	H0 is approved Data variance is stationary/homogenous	H0 is approved Data is stationary/homogenous	H0 is approved Data is random	Lower Limit is not approved Upper Limit is approved

(Source: Calculation Result)

TABLE 3. Data of Average Maximum Daily Rainfall

No.	Year	Average Maximum Rainfall (mm)
1	2009	70,000
2	2010	363,500
3	2017	375,500
4	2015	399,000
5	2016	409,500
6	2011	472,000
7	2012	477,500
8	2018	488,000
9	2013	497,000
10	2014	510,500
Total		4062,500
Mean		406,250
Sd		129,566
n		10

(Source: Calculation Result)

Calculation of Rainfall Plan

The calculation of frequency analysis in this research was aimed to count the plan of rainfall that would be used for input of HEC-RAS program. This research exerted four methods of frequency analysis as normal distribution, normal log, Gumbel, and Log Pearson type III. The probability distribution was approved when the maximum deviation value < critical deviation, as it was shown on table 4. The recapitulation of ΔP maximum value and ΔP critical value would be illustrated below.

TABLE 4. Recapitulation of ΔP Maximum Value and ΔP Critical Value

Probability Distribution	ΔP Maximum Value	ΔP Critical Value	Explanation
Gumbel	1.168	0.410	Disapproved
Normal	0.180	0.410	Approved
Normal Log	0.091	0.410	Approved
Log Pearson Type III	2.567	0.410	Disapproved

(Source: Calculation Result)

Based on the table above, the probability distribution of normal and normal log indicated that the ΔP maximum value < ΔP critical value, while the probability distribution of Gumbel and Log Pearson type III showed that the ΔP maximum value > ΔP critical value, thus, it was summed that only the result of probability distribution test of normal and normal log which could be approved to analyze the rainfall data.

Meanwhile, the recapitulation of calculation result on Chi Square test employed four types of distribution: normal distribution, normal log, Gumbel, and log pearson type III that would be presented on the following table: Table 5. Recapitulation of X^2 value and X^2_{cr} value for four types of probability distributions.

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TABLE 5. Recapitulation of X^2 Value and X^2_{cr} Value for Four Types of Probability Distributions

Probability Distribution	X^2 Value	X^2_{cr} Value	Explanation
Gumbel	11.00	5.991	Disapproved
Normal	8.00	5.991	Disapproved
Normal Log	5.00	5.991	Approved
Log Pearson Type III	4.00	5.991	Approved

(Source: Calculation Result)

Based on the table 5, the probability distribution on Gumbel and normal were not approved, because it indicated the value of $X^2 > X^2_{cr}$. While, the probability distribution of normal log and log pearson type III indicated the value of $X^2 < X^2_{cr}$. Then, it could be concluded that only the test result of probability distribution on normal log and log pearson type III which could be approved to analyze the series of rainfall data.

Based on the table 4, on Smirnov Kolomogorov testing and table 5 on Chi Square testing, this research concluded that the probability distribution that could be approved and has the lowest value was probability distribution of normal log. Further, based on the result lowest value distribution, it would refer to the mean value from each distribution test. How to calculate would be defined below:

1. Smirnov Kolmogorov Test

$$\text{Normal Log} = \frac{0{,}410 - 0{,}091}{0{,}410} = 0{,}778$$

2. Chi Square Test

$$\text{Normal Log } \frac{5{,}991 - 5{,}00}{5{,}991} = 0{,}166$$

Based on the calculation result, it could refer and conclude that the lowest value was acquired from the calculation result of chi square with mean value 0,166, and the lowest value was acquired from the calculation result of Smirnov Kolmogorov test with mean value 0,778.

Analysis on Canal Capacity

The calculation on canal capacity exerted Manning steady flow formula, which the data on this calculation formula was derived from the result canal measurement. The data that would be used in this calculation stage covered to canal length (Ld), canal base width (b), canal depth (h), and canal condition to determine coefficient of manning roughness (n).

Analysis on Linkage Condition of Existing Canal

The analysis on linkage condition of existing canal exerted HEC-RAS software which aimed to ease the making of mathematic analysis model on research location. By benefitting HEC-RAS software, it could identify the depth and speed of flow on each cross section that changed in certain timed. The analysis result of steady flow hydraulics. The following was the result of existing simulation process, the first simulation was primary canal simulation. The flow profile from simulation result would be illustrated on the following figure:

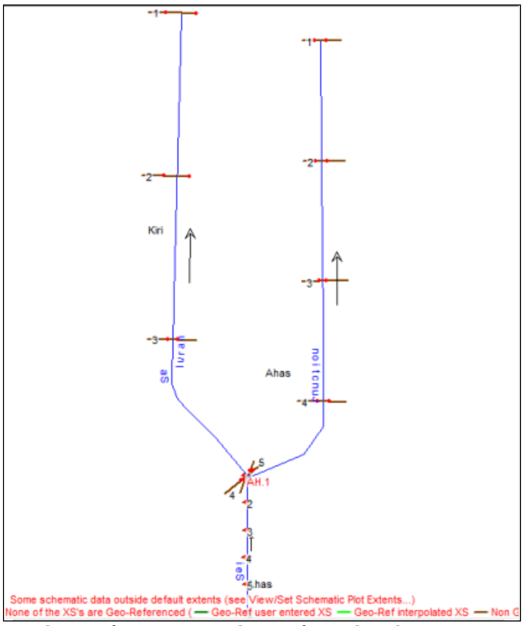

Fig. 4. Scheme of Existing Condition of Canal Linkage on HEC-RAS program

1) First Simulation

The first simulation was on primary canal, it has already existing floodgate. The flow profile from simulation result was shown below:

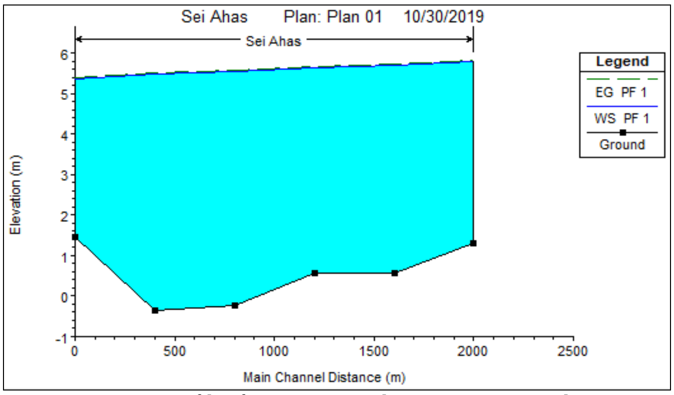

Fig. 5. Profile of Long Section Flow on Primary Canal

TABLE 6. Data of Long Section Simulation Result on Primary Canal

River Sta	Q total (m^3/s)	Min Ch El (m)	W.S. Elev (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m^2)	Top Width (m)	Froude # Chl
1	278.62	1.45	5.34	5.40	0.000409	1.06	274.810	115.42	0.21
2	278.62	-0.35	5.47	5.50	0.000166	0.85	377.580	130.03	0.14
3	278.62	-0.24	5.54	5.58	0.000253	1.09	290.240	92.20	0.18
4	278.62	0.56	5.64	5.67	0.000162	0.79	386.880	136.72	0.14
5	278.62	0.56	5.70	5.73	0.000150	0.83	395.510	136.72	0.14
6	278.62	1.31	5.78	5.83	0.000335	1.06	303.690	130.28	0.20

(Source: Calculation Result)

2) Second Simulation

The secondary canal which was located on left branch did not have existing floodgate. The flow profile from simulation result was illustrated on figure below:

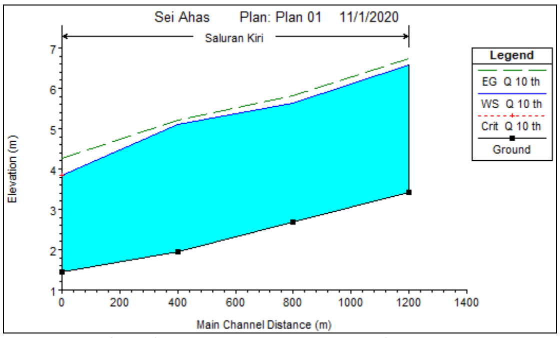

Fig. 6. Profile of Long Section Flow on Left Secondary Canal

TABLE 7. Data of Long Section Simulation Result on Left Secondary Canal

River Sta	Q total (m^3/s)	Min Ch El (m)	W.S. Elev (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m^2)	Top Width (m)	Froude # Chl
1	278.62	1.45	3.83	4.25	0.009735	3.04	100.500	110.38	0.91
2	278.62	1.94	5.11	5.20	0.000994	1.44	209.570	116.55	0.32
3	278.62	2.68	5.64	5.80	0.002314	2.09	162.280	117.75	0.48
4	278.62	3.42	6.58	6.75	0.002388	1.97	159.300	118.92	0.48

(Source: Calculation Result)

3) Third Simulation

The secondary canal on left branch did not have existing floodgate. The flow profile from simulation result was illustrated on the following figure.

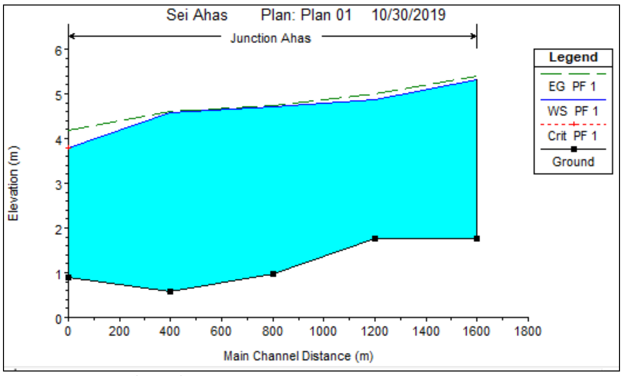

Fig. 7. Profile of Long Section Flow on Right Secondary Canal

TABLE 8. Data of Long Section Simulation Result on Right Secondary Canal

River Sta	Q total (m^3/s)	Min Ch El (m)	W.S. Elev (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m^2)	Top Width (m)	Froude # Chl
1	278.62	0.38	3.79	4.20	0.007884	3.18	108.330	127.50	0.85
2	278.62	0.58	4.57	4.62	0.000351	1.00	308.700	138.58	0.20
3	278.62	0.98	4.71	4.74	0.000288	0.96	337.810	149.66	0.18
4	278.62	1.76	4.88	5.00	0.001655	1.79	198.230	160.79	0.41
5	278.62	1.76	5.32	5.39	0.000651	1.31	237.690	114.58	0.27

(Source: Calculation Result)

From those three process of hydraulic model simulation on each canal, the researchers made an elevation comparison from each canal that was presented on figure 8. The Comparison of Long Section Elevation

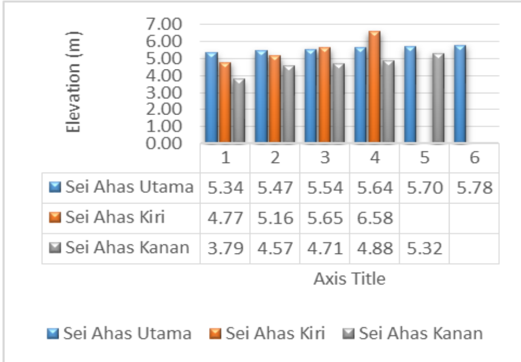

Fig. 8. Comparison of Long Section Elevation on Each Canal

Based on the figure above, the simulation on left secondary canal indicated the highest water level elevation from three canal linkage on the cross section 1-4. While, the water level elevation on primary canal was relatively same and the lowest water level elevation was on right secondary canal.

Water Table Analysis

By analyzing secondary data that had previously been carried out by the Kuala Kapuas Regional I Swamp Hall Office, which is part of the activities of the Peat Restoration Agency which is currently still being carried out. Secondary data obtained are data from the results of field measurements using a Water Level Logger which is then analyzed into MAT elevation data. And from the results of the analysis, a graph of fluctuation shows that the water level in the peat location on the left of the channel ranges from 1.833 cm - 4.751 cm (Figure 9), while in the peat location on the right the channel ranges from 2.870 cm - 7.584 cm (Figure 10).

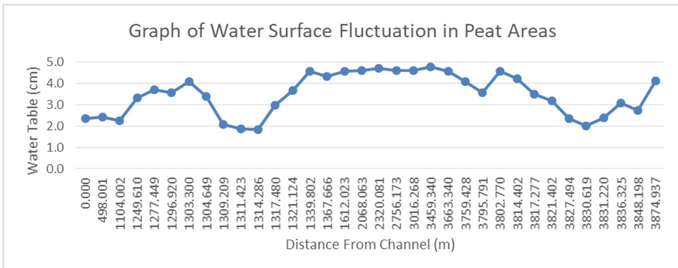

Fig. 9. Graph of groundwater level fluctuation in the peat location to the left of the channel

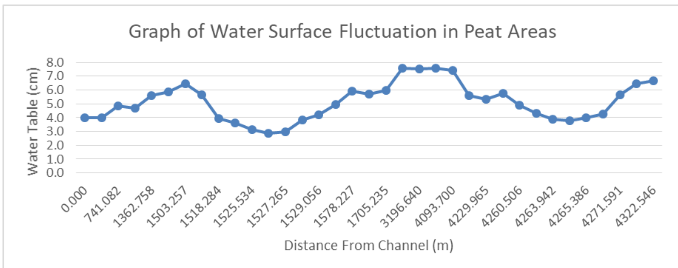

Fig. 10. Graph of groundwater level fluctuation at the peat location to the right of the canal

From the results of field observations, it can be seen that the groundwater level data on the left and right side of the channel fluctuates from various distances from the channel. It

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can be seen that the ground water level gets deeper if it is closer to the drainage channel, although in this case the distance from the measurement point shows a different groundwater level difference as a result of this channel and the construction of cannal blocking. However, water conditions in the upstream area are influenced by the presence of water structures (dams) which cause different heights of about 10 cm - 20 cm from the downstream areas.

Handling of Drought Problems on Research Location

The arrangement of open and close operation pattern of floodgate referred to the step of floodgate closing when the discharge of top tide was coming, and recommendation making for each secondary canal to construct floodgate and the floodgate would be closed during process of tide. This step was employed so the tide discharge that got into primary canal would not return to the river, which could make the water level elevation into the average and no dryness on the canal.

Based on the results of the analysis of previous calculations, an experiment running the HEC-RAS software program was carried out on the secondary channel with the addition of a floodgate on each secondary channel to determine changes in water level elevation after the addition of the floodgates. Following are the results of the trial process of running HEC-RAS on the secondary channel after the addition of a floodgate on the channel, the results of the running are presented in Figure 11 and Figure 12.

Fig. 11. Comparison of Left Secondary Channel Water Level Height with Sluice Gate

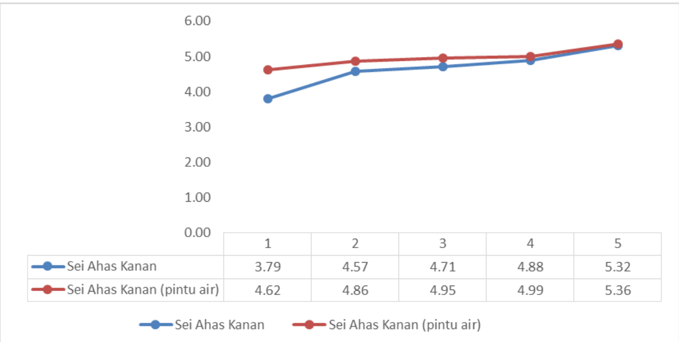

Fig. 12. Comparison of the Water Level of the Secondary Right Channel with the Water Gate

Based on the above calculations, the development that will be carried out is based on the results of the analysis of the water gate operation pattern, it can be concluded that by making 2 floodgates in each secondary channel, which shows the results that the addition of sluices can help increase the height of the water level so that it can meet the the average water level especially in overcoming water shortage in the canal in case of drought in the dry season.

Utilization of Irrigation Water Pump

The irrigation pump was last alternative when the process of tide utilization could not fulfill the water need on land (Kalsim, 2001: 19).

$$q = \sum \frac{A\,x\,y}{R\,x\,T} \times \frac{1000}{36} = \sum \frac{133{,}45 \times 5}{6 \times 10} \times 27{,}78 = 308{,}821 \text{ l/dt}$$

Description:

A : 133,45 Ha

y : 5, 10, 15 and 20cm

R : 6 days

T : 10 hours/day

The calculation with depth (y) 5cm acquired 1 value 308,821 1/dt or 0,30 m3/dt. To achieve the total need of pump with assumption of pump capacity per unit 0,077 m3/dt.

Thus, $= \sum \frac{0{,}30}{0{,}077} = 3{,}896$ or 4 units

Based on the calculation, it required the development through two units of floodgate construction in each secondary canal and fix a unit of floodgate on primary canal, and provide 4 units of water pump.

V. CONCLUSION

Based on the analysis and evaluation result, this research concluded a number of important points from this case study:

1. The evaluation results show that the existing condition of the water level in the primary channel, the average water level can exceed the land elevation but is not maximal, the average inundation height is only 0.001 m or 1 mm, so that land on the right or left side is not inundated. Meanwhile, the evaluation on the left secondary channel and the right secondary channel shows that the average water level has not been able to exceed the land elevation, there is even a decrease in the water level at the downstream side of the channel, so it is necessary to make additional floodgates so that the tidal discharge entering the primary channel does not return to river so that the water level is average and there is no dryness in the channel. Where, the results of the calculation of the peat water level show that there are differences in the level of different groundwater levels as a result of the existence of these channels and the creation of cannal blocking. However, the water condition in the upstream area is influenced by the presence of water structures (dam) which causes a different height of about 10 - 20 cm from the downstream area.
2. Based on the results of running HEC-RAS in the experiment of adding water gate operation patterns in each of the secondary channels that do not have 2 sluice gates for handling raising the water level, the results show that there is an increase in the height of the water level so that it can meet the average water level. especially in

overcoming water shortages in the canals in the event of drought in the dry season. As well, there is also a need for repair and special management for the maintenance of 1 unit of existing floodgate or canal blocking in the primary channel, so that the process of equalizing the water level can run properly.

3. It is necessary to arrange the water system with a closed open water gate operation pattern where by closing the water gate in the primary channel when the peak flow of the tide enters the channel and closing the gates in the secondary channel so that the process of equalizing the water level can run properly. As well as the last alternative to increase inundation height on land between 5-15 cm, dry conditions or no rain for a long time, need to be pumped. If the desired inundation height is 5 cm with an operating time of 10 hours / day for 6 days, then the number of pump units needed is 4 units. What is expected to be able to maintain the surface water level of the canal so that it is 1 m from the surface of the peat and can increase the radius of the wetting of the peat so that it can reach more than 100 meters.

Suggestions

Based on the research finding on this case study, the researcher suggests these following points to related parties:

1. To conduct further analysis which aims to identify the effect of water resistance and effect of ebb and flow that occur from the springhead of primary canal on DAS Kapuas until the crossing or zero point on this research.
2. To conduct further analysis from the aspect of tidal hydrological analysis to determine the effect of the amount of tidal discharge.
3. To construct floodgate on each secondary canal which hold back rainwater discharge or resist tide discharge when the water level in land has been fulfilled.
4. To regulate the pattern of floodgate operation, so all canal linkages can be irrigated and flowed by turning of floodgate opening. To fix and manage specifically for the maintenance of existing floodgate or canal blocking, so they can work and function properly.
5. Further research is expected to be carried out in the dry season or in seasons where the intensity of rainfall is small, because during the dry season the elevation of the ground water level is not influenced by rainfall so that more accurate data is obtained regarding the effect of canal blocking on Groundwater Level fluctuations as an effort to wet peatlands.
6. In next research, more data were analyzed regarding the groundwater level after the construction of the canal block was carried out to obtain more accurate results regarding the effectiveness of the channel block against groundwater level fluctuations.
7. The simulation which is used through HEC-RAS 4.1.0 program is one dimension modeling with its restrictions. To get more maximal result, the modeling can be sustained by exerting model 2 or three dimensions

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