Water Management Evaluation for Upgrading Tidal Irrigation System, Katingan, Kalimantan

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Abstract. Due to channel sedimentation, water deficits and flood still becomes problems in Katingan tidal irrigation system, while the stakeholder plans to reclaim new area in the presence of pyrite soil. This research aims to overcome these problems through four water management scenarios simulated using HEC-RAS to meet the needs of water supply, flushing and drainage. The result shows that water leaching requirement for existing area is 2.48 million m³ and could only be fulfilled by canal normalization. Moreover, extended network with narrow intake canal supplies less than 4.3 million m³ water leaching, so that wide canal connected to the river is required to facilitate more water flowing into the system. Related to flushing, installing pumps in the middle intake could increase canal circulation with one-way outflow during ebb tide, while the inflow depends on flap gates at the same position. Furthermore, the volume of rainwater is generally able to be discharged for every rainy day by all scenario designs. However, existing condition and network expansion with narrow intake canal have less rainwater deficit duration. In conclusion, leaching and drainage aspects require more water storage and wide canal intake, where flushing criterion need the use of pumps and gates.

Keywords: Water Management, Canal Normalization, Reclamation, Tidal Irrigation

1. Introduction

In Indonesia, swamp area is about 36 to 39 million ha, of which about 20 million ha is tidal swamp areas and mostly located on the islands of Sumatra, Kalimantan and Papua. Nearly 4 million ha of this area has been reclaimed, partly by local residents (over 2.5 million ha) and by the central government (around 1.3 million ha). Tidal irrigation area of Katingan I is one of them with approximately 4970 ha of existing area. After a long period of operation, the problem in this area is channel sedimentation leading to reduces the flow capacity and to causes flood and water deficit. On the other hand, land expansion (Program Cetak Sawah) continues to be carried out with the aim of productivity and agricultural products [2]. However, reclaiming new area about 3680 ha in tidal swamps containing pyrite soils requires careful planning, because the reclamation process is relatively long and can contaminate the water and soil quality of the land downstream.
Pyrite soil is often related to the potential for acid sulphate soils or peat soils. Sulfuric acid soil develops as a result of drainage of materials rich in pyrite. In this area, the depth of pyrite ranges from 30-40 cm with a concentration of 30-50%. Meanwhile, the peat forest is located on the eastern side of the network, so that water management must also keep the peat forest water level. Regarding the key to successful management of sulfuric acid soils, water management is the most applicable technique [1]. [6] emphasizes that handling sulfuric acid can be done by reducing the concentration of toxins through leaching using fresh water. Konsten et. al. (1991) in [6] recommends a volume of water for leaching of 5 cm or 500 m³/ha/day. The current area (4970 ha) requires 2485000 m³/day while the area due to expansion (8650 ha) requires 4325000 m³/day. These values are very high compared to the maximum water requirement for planting in a year which is only 8.6 mm so that the water requirement for planting is considered to have been met by fulfilling water leaching.

As a result, there is a need to evaluate the water supply conditions in several water management scenarios to meet the criteria for irrigation and acid dilution. Next to this, flushing of leached water should also be dealt to prevent accumulation of acid water and sedimentation within the system. Regarding the flood, water management must also increase drainage capacity as response to rainwater. Finally, this research is able to provide recommendations for the rehabilitation and operation of the tidal irrigation system of Katingan I.

### 2. Water Management

Some of the main objectives of water management are the discharge of excess water (flooding), the supply of irrigation water, the prevention of dangerous subsidence of groundwater levels, the dilution and flushing acid matter in the field and channels. The drain-ability, irrig-ability and plant species suitability can be determined simply based on hydro-topographic classification (A to D), in which field level is related to the water level from the river, the nearest channel, or the point of intake [5]. 75% of this study area was class A and 25% was class B, by connection to the tides of the Katingan River (semi-diurnal tide type).

As the water leaching supplied, there was a need to flush the water out faster during low tide due to back acid water during high tide [7]. Additionally, flushing could also avoid sedimentation. In the study area, there was an estimated section between the two primary intake channels where the flow rate was very low (death zone) due to the equal flow from different directions. [4] suggested to increase circulation in the system to aid flushing, which can be done with the use of one-way floodgates and pumps.

To prevent flooding, the drainage capacity was calculated using the drainage modulus concept, whose value was calculated by the water balance in extreme conditions and inundation tolerance [3]. The rainwater of 204 mm was designed using 3-days maximum cumulative rain with 5 years return period. The drainage modulus calculation (see Table 1) applied 2 values of inundation permits, for rice field (5 cm) and forest (15 cm) drainage.

#### Table 1. Drainage modulus and its cumulative volume.

| Condition | Area (ha) | Drainage Modulus (lt/s/ha) Field | Total Discharge (m³/s) | Volume (m³) |
|-----------|-----------|----------------------------------|------------------------|-------------|
|           | Field | Forest | Field | Forest | Hourly | Daily | 3-Days |
| Existing | 5093.6 | 7819.2 | 7.9 | 2.1 | 56.66 | 203962 | 4895078 | 14685235 |
| Extension | 8648.5 | 6766.1 | | | 82.50 | 296989 | 7127741 | 21383222 |

On the eastern side of the system, there was a flow (natural river) from outside (forest) affecting water condition in the system with a fairly large catchment area (see Table 1 and Figure 1). This was done by the delineation of digital elevation model along with interpretation of satellite images, showing the presence of several small rivers connected to the irrigation system.
3. Hydraulic Simulation

Flow simulations due to tidal water level fluctuations should be carried out using unsteady flow analysis. The flow simulation on the channel could be solved with the help of the HEC-RAS program, which is able to solve the continuity and momentum equations numerically using the finite difference method with an implicit scheme [7]. Two different times applied to the simulation, neap tide for supply and flushing evaluation; and spring tide for drainage evaluation. There were 4 scenarios applied, where in each scenario there were three different secondary channel designs, namely (a) existing conditions, (b) certain normalization and (c) total normalization. The secondary channel normalization design had a width of 10 m for the existing area (secondary A and B) and a width of 13 m for the expansion area (secondary C). Figure 2 shows the layout along with some points for evaluating the hydraulic properties of the system.

Regarding existing area, all the system in scenario 1 was the existing condition (without expansion), so that the result should meet the water level calibration criterion to review the suitability of the model to field conditions, by adjusting the Manning values. In addition, the rest of simulation for existing area was planned to normalize the primary channel by maintaining the channel width and the lowest water level. Meanwhile, the system in scenario 2 applied new reclamation system, where the primary channels (existing and new) had a similar dimension as in scenario 1.

To increase the flow carrying capacity, scenario 3 was planned with a width of 40 m for primary canal that were directly connected to the river (see Figure 2), namely left, right and middle primary channel in part B (left and right). Then the secondary channel C 1 (left and right) was widened to 30 m to increase the flow capacity to primary D. Concerning to flushing, pumps and flap gates in scenario 4 were designed to increase the flushing of acid water and avoid sedimentation, in addition to depend on supply need. The flap gates functioned to prevent the flow from the side part of the system to enter the middle primary channel during low tide but was still able to supply during high tide. Meanwhile, the pumps functioned to supply fresh water to the left and right sides as the primary channels B, C and D in time the water level started to recede.

However, the pumps were not operated during the rain simulation, assuming that drainage water due to rain was able to dilute acidic water in field and canal and was able to push the flow faster out of the system. For the purpose of evaluating drainage capacity, the input flow due to rain entering the system was represented by the discharge (lateral flow), entering each secondary channel at center point (from the fields) and flowing to the C or D primary channel (from the forest) for 72 hours during spring tide.

Figure 1. Study area and catchment area.
4. The Result

4.1. Calibration
Based on the calibration results, the Manning number for the secondary channel was 0.065 while for the primary channel it varied from 0.02 to 0.06 depended on the cross-sectional area of the channel. The larger wetted perimeter of cross section, the smaller the Manning figure. Two evaluation points (P01 and P04) were used to review the comparison of water levels (see Figure 3). The difference in water level was quite large due to the presence of channels outside the network that were not measured and were not modelled. In addition, during hydrometric measurements, rains may have occurred and affected the water level.

4.2. Pumps and Gates
The gate and pump designs in scenario 4 determined the supply and flushing capability of the system. By considering the need for water leaching, four flap gate locations on primary channels B and C installed with four gates while on primary channel D (2 locations) only applied 3 gates. The size of each door was 2x2 m with a base elevation of 2.5 +m and would only open when the water level in the middle primary channel is higher than the channels on the left and right. Regarding to flushing at low tide, the pump capacity was 4.5 m³/s operating during low tide for 12 hours. Figure 4 shows the discharge at the combination of sluice and pump along with the water level of the middle primary channel for 3 tidal cycle. At high tide, the gates of the primary channels B and C flowed 10 to 20 m³/s, while primary D was only about 9 m³/s. A significant difference occurs at the gates in the primary channel C, where the right side flowed more water (20 m³/s) than the left side (13 m³/s). This was because the service area on the left was larger than the right side.

Figure 2. System layout and important points.
The pump capacity and gate dimension could later be redesigned as the need for leaching and flushing may vary. Alternatively, if the pump and gate are to be operated at different times, increasing the dimensions of the gates will help increase the water supply at high tide, while increasing the pump capacity will intensify the flow to avoid sedimentation. This flexibility benefits the stakeholder to consider financial aspect to develop the irrigation system.

**Figure 3.** Water level between measurement and computation.

**Figure 4.** Pump and gate discharges.
4.3. Supply

Table 2 and Figure 4 shows the maximum and minimum cumulative volume entering the system. With water demand in the current irrigation area (2485000 m$^3$), the condition of the existing network (scenario 1) was not able to meet the supply criteria with only 70.2% of the incoming water volume of the total demand. Meanwhile, with the rehabilitation of secondary canals as in scenarios 1-a, 1-b and 1-c, the cumulative volume of incoming water exceeded 2.5 million m$^3$.

In the system with extended area, the channel design in all of scenario 2 did not fulfill the required volume of 4.325 million m$^3$. Although normalization was carried out on all channels (including secondary), the maximum value achieved was only 3.3 million m$^3$. This indicates that even the system had large water storage, the intake capacity was not large enough to flow in the water. Hence, a larger or wider primary intake channel was needed.

Table 2. Cumulative inflow volume among scenarios.

| Scenario Geometries | Water Needs (m$^3$) | Cumulative Volume (m$^3$) | Surplus/Deficit (m$^3$) | %    |
|---------------------|---------------------|---------------------------|-------------------------|------|
|                     | Minimum             | Maximum                    |                         |      |
| Scenario 1          | 2,485,000           | 59,540                     | -1,744,238              | -740,762 | 70.2% |
| Scenario 1-a        | 86,901              | 2,674,333                  | 189,333                 | 107.6% |
| Scenario 1-b        | 105,862             | 2,808,228                  | 323,228                 | 113.0% |
| Scenario 1-c        | 217,629             | 2,910,625                  | 425,625                 | 117.1% |
| Scenario 2-a        | 4,325,000           | 156,773                    | -2,949,599              | -1,375,401 | 68.2% |
| Scenario 2-b        | 168,757             | 3,131,996                  | -1,193,004              | 72.4% |
| Scenario 2-c        | 195,649             | 3,376,429                  | -948,571                | 78.1% |
| Scenario 3-a        | 251,293             | 4,312,008                  | -12,992                 | 99.7% |
| Scenario 3-b        | 274,133             | 4,473,423                  | 148,423                 | 103.4% |
| Scenario 3-c        | 397,762             | 4,670,844                  | 345,844                 | 108.0% |
| Scenario 4          | 191,642             | 4,397,956                  | 72,956                  | 101.7% |

All scenario 3 designed to widen primary intake channel increased the flow entering the system during high tide. The volume of incoming water reached 4.6 million m$^3$. Even without rehabilitation of secondary canals, the percentage of inflow water compared to the needs was almost achieved (99.7%). Similarly, the combination of pumps and flap gates in scenario 4 was able to supply more than 4.4 million m$^3$ of water in one tidal cycle.

Figure 5. Cumulative volume of incoming water.
4.4. Flushing
The cumulative volume of incoming water showed in Table 2 had a minimum value above zero, so that there was still an amount of acid water pushed back into the system before being drained. In scenario 3, the minimum value was 397762 m$^3$ and considered to be high. For this reason, the left and right primary channels (intake) shall be functioned primarily as drainage channel while the middle primary shall focus on water supply despite still having two-way flow. This was done by installing pumps and flap gates as in scenario 4, thus the minimum cumulative volume could drop to a half of the value in scenario 3. Figure 5 shows that in scenario 4, the inflow water was almost the same between the middle and the side primary intake (left and right) but was drained more through the side primers. With this one-way flow at low tide, the process of leaching acid water would be much better.

![Figure 6. Incoming water at different intake.](image)

In addition to the importance of diluting acid water, the design in scenario 4 was also intended to avoid sedimentation in the primary channel (middle part), with higher flow compared to the design in scenario 3. Figure 6 shows the discharge at the end of the primary channels B, C and D (discharge control point). At high tide, the discharge between scenarios 3 and 4 were relatively similar at the same location. However, at low tide in the primary channel B, the use of pumps in scenario 4 enhanced the higher discharge by about 5 to 7 m$^3$/s. Meanwhile, for primary channel C, the rise was around 4 to 6 m$^3$/s. A relatively small difference took place in the primary channel D, where in scenario 3 the value was lower by approximately 2 m$^3$/s. Thus, the pumps and gates could aid the primary channel be more stable from the risk of sedimentation.

4.5. Drainage
The water balance within the system detailed in figure 7 informs that in general the system drained rainwater properly, in which the volume of water flowed out at low tide was relatively greater than the rainwater volume dropped to the system. However, the duration of deficit rainwater volume in scenarios 1, 2-a, 2-b and 2-c was less than 12 hours, so that more than 12 hours of water surplus within the system occurred in a day.
Figure 7. Discharge at end-part channel of primary B, C dan D.

Although the circulation of system was good enough to drain the rainwater, the attention should be paid to the water drained from the forest in the east side. This is because mostly the forest contains peat soils, in which land subsidence and degradation could be faced as a result of lowering peat water table. Hence, it is suggested that canal blocking technique be applied to minimize drained water from the forest. This also could reduce the amount of water that should be flowed out by the system.

Figure 8. Water balance during rainy days.
5. Conclusion
In areas with shallow pyrite content such as the irrigation area of Katingan I, channel widening was preferable to deepening in the aim of reducing acid sulphate soils oxidation. As the need to facilitate more water entering the system, the wider primary channels which are directly connected to a river or water source must be carried out. Even though water storage in the system is quite large, the incoming water volume will not be sufficient if the intakes are not a large enough. The discharge of acid water from leaching (flushing) must also be streamlined by implementing a one-way drainage to minimize acidic water pushed back into the system. This could be done by lining the channel supported by the pump and one-way floodgate installation. In addition, this alternative could also reduce the risk of sedimentation by increasing the flowrate and transforming the flow direction at low tide.

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