Alternatives of flood control for the Linei river, city of Toboali (a case study of the Rawabangun region)

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Abstract. Rawabangun is one of the areas in the City of Toboali in South Bangka Regency that is often flooded. There is a main river that passes through the Rawabangun area, which is the Linei River. The aim of this study is to analyse alternatives that can be used in flood control in Rawabangun. The analysis was carried out with the 5.1 Storm Water Management Model (SWMM) and 5-year planned rainfall. There were 9 overflow points found on the channel, and the implementation of flood control is to be carried out with infiltration wells, retention ponds, and channel modification. The application of infiltration wells with a maximum of 1267 units can eliminate 8 overflow points. The application of 3 units of retention ponds with a maximum of 3 units can eliminate 5 overflow points. Changing the channel depth to 4 m can eliminate all overflow points but is not relevant to be applied.

Keywords: Flood, Flood Control, Reduction, SWMM 5.1, Rawabangun Region.

1. Introduction

Flooding is a common problem that occurs in most parts of Indonesia, especially in densely populated areas such as in urban areas. Therefore, the losses caused can be very large in terms of both material and losses of life. Currently, it is appropriate that the problem of flooding has received serious attention and is a problem for all people. Assuming that the problem of flooding is a common problem, it should be necessary that various parties to pay attention to the matters that can cause flooding and anticipate them as early as possible to minimize the losses incurred [1].

Toboali, the capital of South Bangka Regency, is one of the areas in Bangka Belitung that has been hit by floods. One of the areas in Toboali that is often flooded is the Rawabangun area. There is one main river, called the Linei River, in the Rawabangun area, and wastewater from almost all drainage in Toboali ends up in this river. The Rawabangun area in Toboali is at an altitude of ± 5 metres above sea level [2]. The distance between Rawabangun and the sea area is only approximately ± 1 kilometre, and thus the tides are thought to have an effect on the outlet process of wastewater in the downstream part of the Linei River. When there is excess water due to rain in the river channel, it is suspected that water cannot flow out because it is held back by sea tides, and the result is that excess water will inundate low areas in the surrounding area. This situation forces the residents of the Rawabangun area to must always be alert in the event of rains in Toboali Sub-District with high and medium intensity and within a certain period [2].
In a previous research conducted by James Andrew Griffith et al. (2019) regarding flooding due to the influence of sea tides, the sea level rise (sea tide) will increase urban flooding [3]. In another study conducted by Try Al Tanto (2014), who examined flood events in the coastal area of Bungus in the City of Padang, it was found that the flooding that occurred in the area was caused by tides of sea water and the height of inundation at some points was less than 0.5 to 2.5 metres [4]. If rain occurred with high intensity accompanied by sea tides, then it can be ascertained that the area in the elevation range will be flooded, either by seawater or by water in the drainage system on the land. Based on these two studies, it is suspected that the flood conditions that occur in Rawabangun and surrounding areas are one of the effects of the presence of sea tides that enter the Linei River channel. From the background description, the following are the problems in this study. First, what is the current condition of the Linei River network and existing drainage? Second, what are the alternatives for flood control that are in accordance with the Linei River area, especially Rawabangun and surrounding areas? Third, how can alternative solutions be applied in order to reduce flood discharges that occur?

2. Materials and Methods
The location of this study is the City of Toboali, located in Toboali Sub-District in South Bangka Regency, Province of Bangka Belitong Islands. The geographical location of South Bangka Regency is within 2° 26’ 27” - 3° 5’ 56” South Latitude and 107° 14’ 31” -105° 53’ 09” East Longitude. The area of the Linei River catchment area (CA) is 412 ha.

![Figure 1. Map of Study Location of Liner River Catchment Area.](image)

2.1. Material
The analysis in the study was carried out with the assist of the Storm Water Management Model (SWMM) software.

2.1.1. Storm Water Management Model (SWMM)
SWMM is a dynamic rainfall runoff simulation model for single events or long-term (continuous) simulations, especially in urban areas. The output of SWMM is the quantity of runoff, flow rate, flow depth, and quality of runoff produced in each sub-catchment. SWMM 5 had recently been developed for hydrological modelling of several types of LID (Low Impact Development). Operating on Windows, SWMM 5 provides an integrated system for editing drainage area input data; running hydrological, hydraulic, and water quality simulations; and showing results in various formats. The
software includes features of colour-coded maps of drainage areas, time-series graphs and tables, profile plots, and statistical frequency analysis.

2.2. Research Methodology

2.2.1. Data Collection

Some data needed to be prepared for this study. The SWMM model works by entering the required data into the program. More complete input data results in a better construction of the SWMM model. The data needed include daily rainfall data, data of drainage and river network, geometry data of drainage and river channels, land/soil data and topographic data, land use data, data on groundwater depth, and data on highest tides.

2.2.2. Analysis of Design Rainfall

Before the rainfall data could be used for the analysis, testing was performed on rainfall data. Testing of rain data was performed by statistical analysis. The statistical analysis involved, among others, tests of consistency, outlier-inlier, stationary, persistence, and no trend. The planned rainfall used in this study is rainfall with a 5-year return period. The following is the process of rainfall data analysis:

a. Point Rain

At the study location, there was only 1 rain station, and thus the rainfall used in this study was taken based on daily rainfall data from Rias Station.

b. Planned Rainfall

Planned rainfall in the study was analysed using Log Pearson III, Log Normal and Gumbel. Frequency analysis showed that only the statistical parameters of Log Pearson III met the criteria, and thus the planned rainfall analysis utilized the Log Pearson III method. The Log Pearson III method utilized the following equation [5]:

\[ \text{Log } X = \text{Log } \bar{X} + k (S\text{Log}X) \]  

(1)

Where Log \( X \) is the logarithmic value of design rainfall; Log \( \bar{X} \) is the logarithmic average value; SLogX is the standard deviation value of \( \log x \); and \( k \) is the Log Pearson Type III distribution constant. In this study, the utilized planned rainfall has a return period of 5 years.

c. Rain Intensity

After having obtained a planned rainfall with a 5-year return period, the next step was converting the data to rain intensity. Rain intensity is the amount of rainfall expressed in the height/depth of rain (mm/h) per unit of time, which occurs at a single time duration, when rainwater is concentrated. The amount of rainfall intensity varies depending on the duration and frequency of rainfall. One common formula for calculating the intensity of rainfall is the Mononobe formula. This formula is often used in calculating rain intensity in urban areas [6]:

\[ Ri = \frac{R24}{t} \left( \frac{t}{7} \right)^{2/3} \]  

(2)

\[ R_T = T \cdot Ri - (t - 1) \cdot R_{(t-1)} \]  

(3)

Where \( Ri \) is the rain intensity during concentration time (mm per hour), \( R24 \) is the daily maximum rainfall in 24 hours (mm), \( t \) is the duration of rain (hours), \( T \) is the concentration time (hours), and \( RT \) is rainfall up to T-hour (mm per hour).

2.2.3. Construction of Existing Models

The model was built with the help of Storm Water Management Model (SWMM) 5.1 software. The existing model was calibrated using the Root-Mean-Square Error (RMSE) method. The variable used as the calibration number is the channel discharge in the model with the observation channel discharge in the field. The calibration is said to be good if the RMSE value is close to zero. A smaller calibration number means that the existing model approach represents the field conditions more closely.
2.2.4. Evaluation of Existing Drainage and River Networks
Evaluation of existing drainage and river networks used the SWMM model. From this evaluation, the points on the conduit (channel) and junction (node) where their capacities are exceeded (to flooding) are identified.

2.2.5. Alternatives of Flood Control
The alternatives of flood control for this study among others include the application of:

a. Retention Pools
   Retention pools are to be placed in open areas or land that is still adequate in accordance with the planned size.

b. Infiltration Wells
   Infiltration wells are to be placed in grounds of homes of residents, with the input being runoff water discharge from the roofs of each house. Theoretically, according to Sunjoto (2011), the volume and efficiency of infiltration wells can be calculated based on the balance of water entering the well and the water that seeps into the soil [7]. Rainwater discharge on the roof and dimensions from the construction of infiltration wells with side walls and empty fixed spaces can be calculated by:

\[ Q = C \times I \times A \]  
\[ H = \frac{Q}{FR} \left( 1 - e^{\frac{-FR}{\pi R^2}} \right) \]  
\[ F = \frac{L + 2R}{2R} \ln \left[ \frac{1 + \left( \frac{L}{2R} \right)^2}{2} \right] \]

Where \( Q \) is input water discharge (m\(^3\)/hour), \( C \) is the roof drainage coefficient (0.95), \( I \) is the rain intensity (m per hour), \( A \) is the roof area (m\(^2\)), \( R \) is the radius of the well (m), \( H \) is the depth of well (m), \( F \) is the geometric factors of the well (m), and \( K \) is the soil permeability coefficient (m/h).

c. Channel Dimension Changes
   For the dimensions of the existing channel, changes are to be made to the depth of the drainage and river channel at the study location. In summary, the alternatives of flood management in this study are implemented by applying several scenarios that can be seen in the table below:

| No. | Alternatives of Flood Control                                      |
|-----|-------------------------------------------------------------------|
| 1   | Construction of 3 retention pools (SU1+SU2+SU3)                   |
| 2   | Construction of 1267 units of infiltration wells                  |
| 3   | Changing existing channel dimensions to a new depth (4 meters)   |

3. Results and Discussion

3.1. Hydrological Analysis
The Linei River catchment area has an area of 412 ha. The closest rain gauge station from Linei River catchment area is the Rias rain station, the only one in the area. The data can be used if the data series has passed statistical tests and is declared feasible. The rainfall data can be seen in Table 2.

The rain data in Table 2 was declared feasible after performing tests of statistical analysis, including tests of consistency, outlier-inlier, stationary, persistence, and no trends. The analysis of the planned rainfall was carried out by the Log Pearson Type III method. From the analysis of rain data with Log Pearson III frequency analysis, the planned rainfall for return periods of 2, 5, 10, 20, 25, 100, 200, and 1000 years was obtained. The planned rainfall obtained from the analysis can be seen in Table 3.

After the planned rainfall was obtained, further testing was conducted with goodness-of-fit testing of frequency distribution. The goodness-of-fit tests of frequency distribution that were performed were the Kolmogorov-Smirnov and Chi-Square tests [8].
Table 2. Rainfall of the Linei River Catchment Area

| No. | Year | Rainfall (mm) |
|-----|------|---------------|
| 1   | 2007 | 148.6         |
| 2   | 2008 | 107.1         |
| 3   | 2009 | 92.0          |
| 4   | 2010 | 124.7         |
| 5   | 2011 | 87.0          |
| 6   | 2012 | 108.4         |
| 7   | 2013 | 128.4         |
| 8   | 2014 | 137.6         |
| 9   | 2015 | 141.4         |
| 10  | 2016 | 183.9         |

Table 3. Planned Rainfall

| Tr (Year) | P_{Tr} (%) | X_{Tr} (mm) |
|-----------|------------|-------------|
| 2         | 50         | 123.289     |
| 5         | 20         | 148.964     |
| 10        | 10         | 165.254     |
| 25        | 4          | 184.491     |
| 50        | 2          | 198.151     |
| 100       | 1          | 212.529     |
| 200       | 0.5        | 224.812     |
| 1000      | 0.1        | 254.601     |

According to Suripin (2004), if short rainfall data is not available and there is only daily rainfall data, then the rainfall intensity can be calculated using the Mononobe formula [9]. Then, according to Suhardjono (2015), the rainfall used as the basis for calculating the drainage system for the typology of medium cities with an area of 101 - 500 ha is the rainfall intensity with a return period of 2-5 years. The Log Pearson Type III distribution resulted in a planned rainfall value with a 5-year return period equal to 148.964 millimetres per day.

Table 4. Distribution of Rainfall Intensity with a 5-year Return Period

| No. | Hour | Ratio (%) | Hourly rain intensity (mm) 5 year return period |
|-----|------|-----------|-----------------------------------------------|
| 1   | 1    | 0.550     | 81.978                                        |
| 2   | 2    | 0.143     | 21.308                                        |
| 3   | 3    | 0.100     | 14.947                                        |
| 4   | 4    | 0.080     | 11.899                                        |
| 5   | 5    | 0.067     | 10.048                                        |
| 6   | 6    | 0.059     | 8.784                                         |

After the hourly rainfall intensity value of the planned rainfall was obtained, the rainfall intensity could be used in making the SWMM model.

3.2. Existing Condition Modelling with SWMM

a. Modelling of the existing condition

Existing condition modelling was carried out to determine the condition of the drainage and river channels at the study site before flood control is carried out. The inputs for this existing model are in
the form of data on the 5-year return period rainfall intensity, types of land cover in the Linei River catchment area, the dimensions of drainage channel and rivers at the study location, and the sea tide data in the downstream part of the river.

b. Calibration of existing models

Calibration is the verification process to determine and adjust the correctness of the simulation results with the actual conditions in the field. After the existing model was built based on field conditions, calibration was then carried out for the model. In this study, calibration was performed using the Root-Mean-Square Error (RMSE) method. Model calibration was performed by comparing the discharge on the channel in the field with the discharge of the simulation results. The discharge used as comparison data for calibration is the channel discharge data at the location or sample observation point during the rain on January 9, 2019.

\[
\text{RMSE} = \sqrt{\frac{1}{n} \sum_{i=1}^{n} (Q_{\text{obs}} - Q_{\text{sim}})^2};
\]

\[
= \sqrt{\frac{1}{10} \sum_{i=1}^{10} (0.069813)}
\]

\[
= 0.0835542 \approx 8\%
\]

Table 5. Calculation of Calibration with the RMSE Method

| Obs Channel | Q (m³/s) | (Qobs-Qsim)² | RMSE    |
|-------------|---------|--------------|---------|
| Channel C11 | Sim     | Obs          |         |
| 0           | 0.03    | 0.0009972    |         |
| 0           | 0.06    | 0.0041179    |         |
| 0           | 0.10    | 0.0095699    |         |
| 0           | 0.13    | 0.0175819    |         |
| Channel C11 |         |              | 0.0835542 |
| with width = 3m and h = 1.5m | 0.01 | 0.15 | 0.0205662 |
| 2.51 | 2.54 | 0.0009195 |
| 5.74 | 5.82 | 0.0069772 |
| 7.29 | 7.25 | 0.0016000 |
| 6.92 | 6.93 | 0.0002081 |
| 5.65 | 5.74 | 0.0072751 |

\[
\Sigma = 0.0698130 \quad n = 10
\]

3.3. Evaluation of the Conditions of Existing Drainage and Rivers

After the existing model was calibrated, an evaluation of the existing drainage channel and river was carried out using the planned rainfall intensity with the 5-year return period. From the evaluation results, it was found that there are 9 overflow points on the channel. The details of channel overflow can be seen in Table 6.

3.4. Alternatives of Flood Control

There are three alternatives for flood control that are to be applied at the research location. The three flood control alternatives are infiltration wells, retention ponds, and changes to the dimensions of existing channels.
3.4.1. Retention Ponds

The retention ponds applied in this study are retention ponds on the river body. The data of dimensions and spillway structures for the ponds can be seen in Table 7 and Table 8. The placement locations of the 3 retention pond units can be seen in Figure 2.

Table 6. Channel Simulation Results in Overflow Conditions

| No. | Channel Name | Channel width (m) | Channel depth (m) | Maximum discharge (m³/s) | Information |
|-----|--------------|------------------|------------------|--------------------------|-------------|
| 1   | C7           | 3                | 1.50             | 9.446                    | Overflow    |
| 2   | C8           | 3                | 1.50             | 8.887                    | Overflow    |
| 3   | C9           | 3                | 1.50             | 8.492                    | Overflow    |
| 4   | C10          | 3                | 1.50             | 6.486                    | Overflow    |
| 5   | C11          | 3                | 1.50             | 8.505                    | Overflow    |
| 6   | C14          | 3                | 1.50             | 6.839                    | Overflow    |
| 7   | C41          | 2                | 0.80             | 3.512                    | Overflow    |
| 8   | C52          | 2                | 0.80             | 2.206                    | Overflow    |
| 9   | C55          | 3                | 1.50             | 9.345                    | Overflow    |

Table 7. Retention Pool Dimension

| No. | Retention Name | Pool Dimension Plan | Pool bottom elevation (m) |
|-----|----------------|---------------------|---------------------------|
|     | L (m) | W (m) | H (m) |                       |
| 1   | SU1   | 450   | 30    | 3                      |
| 2   | SU2   | 250   | 30    | 3                      |
| 3   | SU3   | 250   | 30    | 3                      |

Table 8. Spillway Dimension

| Weir ID | Inlet Node | Type     | Height (m) | Length (m) | Inlet offset (m) | Discharge coef. |
|---------|------------|----------|------------|------------|------------------|-----------------|
| Reg1    | SU1        | Transverse | 0.6        | 6          | 2.4              | 1.5             |
| Reg2    | SU2        | Transverse | 0.5        | 5          | 2.5              | 1.5             |
| Reg3    | SU3        | Transverse | 0.5        | 5          | 2.5              | 1.5             |

Figure 2. Placement Locations of Retention Ponds
3.4.2. Infiltration Wells

The type of soil at the study location to a depth of > 3 m is fine silt sand, brownish grey with a permeability coefficient value of 3.6 centimetres per hour. The infiltration wells are planned to have a diameter of 1.2 m and a depth of 3 m. Calculation of infiltration wells utilized a planned rainfall with a 5-year return period equal to 148.96 millimetres per day, and if \( t_e \) is the effective rainfall duration using the formula from SNI 03-2453-2002, then [10]:

\[
\begin{align*}
\text{te} &= \text{effective rainfall duration (hour)} = 0.90 \times 148.96^{0.92} \\
\text{te} &= 0.90 \times \frac{148.96^{0.92}}{60} = 1.497 \text{ hour}
\end{align*}
\]

By using the Mononobe method rain intensity equation, then:

\[
\begin{align*}
I &= \frac{R_o}{24} \times \left( \frac{24}{t_e} \right)^{\frac{2}{3}} = \frac{148.96}{24} \times \left( \frac{24}{1.497} \right)^{\frac{2}{3}} \\
I &= 39.49 \text{ mm/h} = 0.0395 \text{ m/h}
\end{align*}
\]

- **Calculation of the geometry factor of the well (F)**
  
  \[
  L = 2 \text{ m} \\
  K = 0.036 \text{ m/h} = 0.001 \text{ cm/sec} = 0.00001 \text{ m/s}; H = 3 \text{ m}; R = 0.6 \text{ m} \\
  T = 1.497 \text{ h} = 5390.361 \text{ sec}; T = 0.90 \times \frac{R^{0.92}}{60} \text{ (SNI: 03-2453-2002)} \\
  F = \frac{2\pi L + 2\pi R \ln 2}{2R} = \frac{2(2\times2) + (2\times0.6 \times \ln(2))}{2R} \\
  F = 9.936 \text{ m}
\]

- **Calculation of the roof runoff discharge (Q_{roof})**
  
  \[
  Q = C \times A \\
  T = 1.497 \text{ hour}; I = 0.0395 \text{ m/h}; A = 154 \text{ m}^2; C = 0.95 \\
  Q = 5.77788 \text{ m}^3/\text{hour} \approx 0.0016 \text{ m}^3/\text{s}
\]

- **Calculation of the depth of the planned wells, with Q_{roof} = 5,77788 m³/hour:**
  
  \[
  H = \frac{Q}{FK \left( 1 - \exp \left( \frac{-FKT}{\pi R^2} \right) \right)} \\
  H = \frac{5.77788}{9.936 \times 0.036 \left( 1 - \exp \left( \frac{-9.936 \times 0.036 \times 1.497}{\pi (0.6^2)} \right) \right)} \\
  H = 6.0956 \text{ metres}
\]

- **Calculation of the ability of infiltration wells for the roof of 1 house – if the planned depth of the infiltration wells is 3 meters, then:**
  
  \[
  n = \frac{H_{\text{planned}}}{H} \\
  n = \frac{6.0956}{3} = 2.03 \approx 2 \text{ wells for catchment of 1 roof with an area of 154 m}^2
\]

The maximum number of infiltration wells in this study is 1267 units.

3.4.3. Changes to Channel Dimensions

Changes to channel dimensions are to be made only for the depth of the channel, in consideration of the state of the study location. The summary of changes to the channels can be seen in Table 9.
Table 9. Summary of Channel Dimension Changes

| No. | Channel Name   | Shape       | Old height (m) | New height (m) | Old width (m) | New width (m) |
|-----|----------------|-------------|----------------|----------------|---------------|---------------|
| 1   | C7 – C22       | Rectangular | 1.5            | 4              | 3             | 3             |
| 2   | C41            | Rectangular | 0.8            | 4              | 2             | 2             |
| 3   | C51 – C52      | Rectangular | 0.8            | 4              | 2             | 2             |
| 4   | C55            | Rectangular | 1.5            | 4              | 3             | 3             |

3.4.4. The Ability of Flood Control Alternatives in Reducing Flood Points

After all flood control alternatives were implemented and simulated, the capabilities of each alternative were obtained. The summary of flood reduction capabilities of each alternative can be seen in Tables 10 and 11.

Table 10. Effect of Application of Flood Control Alternatives in Reducing Flood Points

| No. | Channel Name | Channel Width (m) | Channel Depth (m) | New Channel Depth (m) | Retention Ponds | Infiltration Wells | Changes to Channel Depths |
|-----|--------------|-------------------|-------------------|-----------------------|-----------------|--------------------|--------------------------|
| 1   | C7           | 3                 | 1.50              | 4.00                  | Sufficient      | Sufficient         | Sufficient               |
| 2   | C8           | 3                 | 1.50              | 4.00                  | Sufficient      | Sufficient         | Sufficient               |
| 3   | C9           | 3                 | 1.50              | 4.00                  | Overflow        | Sufficient         | Sufficient               |
| 4   | C10          | 3                 | 1.50              | 4.00                  | Overflow        | Sufficient         | Sufficient               |
| 5   | C11          | 3                 | 1.50              | 4.00                  | Overflow        | Sufficient         | Sufficient               |
| 6   | C14          | 3                 | 1.50              | 4.00                  | Overflow        | Overflow           | Sufficient               |
| 7   | C41          | 2                 | 0.80              | 4.00                  | Sufficient      | Sufficient         | Sufficient               |
| 8   | C52          | 2                 | 0.80              | 4.00                  | Sufficient      | Sufficient         | Sufficient               |
| 9   | C55          | 3                 | 1.50              | 4.00                  | Sufficient      | Sufficient         | Sufficient               |

Table 11. The Ability of Flood Control Alternatives in Reducing Flood Points

| No. | Flood Control Alternative                                                                 | Existing Flood Points | Remaining Flood Points | Reduction (%) |
|-----|-------------------------------------------------------------------------------------------|-----------------------|------------------------|---------------|
| 1   | Construction of 3 Retention Pools (SU1+SU2+SU3)                                             | 9                     | 4                      | 55.56         |
| 2   | Construction of 1267 Units of Infiltration Wells                                            | 9                     | 1                      | 88.89         |
| 3   | Changing Existing Channel Depth Dimension to 4 metres                                       | 9                     | 0                      | 100           |

Based on Tables 10 and 11, the flood control alternative that is able to reduce all flood points is to make changes to the depth dimensions of existing channels to 4 meters. However, the application of these alternatives is highly irrelevant to the condition of the study location. The application of changes to the depths of the existing canals will be easily influenced by changes in the conditions of the catchment in the future. The changes in question may involve sedimentation, which will make the channel shallower. If the sedimentation at the study site is categorized as high, the water discharge in the channel will overflow in a short time, making the application of this alternative ineffective. The application of retention ponds and infiltration wells in the study location is considered to be more effective and efficient because it is more environmentally friendly, although based on Tables 10 and
11 above, the application of both has not been able to eliminate all flood points. Infiltration wells are able to absorb water into the ground, becoming a ground water reserve. Retention ponds can withstand longer flood peaks that will enter the river channel, giving the river more time to discharge water into the sea.

4. Conclusions
The following are the conclusions that can be drawn from this study:
1. In general, the existing conditions of the Linei River and drainage channels at the study location have been unable to accommodate water rainfall discharge with a 5-year return period. There are 9 channels that exceed capacity and were identified as the overflow points.
2. The following are the recommendations for flood control in the study location based on data of groundwater depth, soil permeability, land use, and density of surrounding buildings:
   ➢ Retention ponds
     - The maximum number of retention ponds to be applied is 3 units:
       o Storage unit 1 (SU1) with a length of 450 meters, a width of 30 meters, and depth of 3 meters
       o Storage unit 2 (SU2) with a length of 250 meters, a width of 30 meters, and a depth of 3 meters
       o Storage unit 3 (SU3) with a length of 250 meters, a width of 30 meters, and a depth of 3 meters
   ➢ Infiltration wells
     - Design: infiltration wells of circular shape with a diameter of 1.2 meters, a depth of 3 meters, and construction with walls made of stone pairs without plaster
     - The maximum number to be applied in the Linei River catchment area is 1267 units.
   ➢ Changing the existing channel depth to 4 meters; the length of the change dimension is ± 500 meters to the upstream when viewed from the outfall.
3. The application of 3 units of retention ponds with a maximum of 3 units can eliminate 5 points of overflow. Application of infiltration wells with a maximum of 1267 units can eliminate 8 points of overflow. Changing the channel depth to 4 m can eliminate all overflow but is not relevant to be applied. Thus, the flood control alternatives are still not able to eliminate all flood points.

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