Study of inundation control on Kelayan Regional Inundation Handling Unit in Banjarmasin, South Borneo

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Abstract. Inundation occurred in high density urban areas is a major issue especially in low land because it potentially damage the affected infrastructure. In this research, a case was taken in Kelayan Regional Inundation Handling Unit in Banjarmasin City, South Borneo. The purpose of this study is to determine cause of the inundation and provide recommendation for reduced it. The hydrological analysis was calculated by using the rational method, using data from Sungai Tabuk Rain Station that 24 years long. From the discharge obtained, then the required channel dimensions could be calculated and compared with the existing channel dimensions. The hydraulic analysis was performed by HEC-RAS model with the discharge as boundary condition, and topographic data to determine channel elevation. As the results, the existing drainage channels can only cover 39.85% of the entire area, and almost all of these existing channels cannot drain the discharge that occurs, therefore it becomes an inundation. For example, channel number 4.C.14c the existing channel has 0.5m width and 0.7m depth, it caused inundation as high as 39cm upstream. After normalized the channel by increasing the dimensions to 2.1m width and 1.3m depth, the discharge does not overflow so there is no inundation.

1. Introduction
The City of Thousand Rivers is a predicate that has been given to the capital of South Kalimantan Banjarmasin City. It is because of the many rivers that flow in the city [1]. There are two main rivers, namely the Barito River, which is the largest river in South Kalimantan, and the Martapura River, which is a tributary of the Barito River.

Conventional drainage systems with the principle of disposing of the stormwater as fast as possible into the river to drain the land are still applied in this city. Because the type of soil is swamps in lowland so that the surface of the ground is always flooded. The soil cannot absorb excess water due to rain. Therefore rainwater becomes runoff, which must be immediately drained into the river. The lack of channel maintenance also made sedimentation very thick and reduced channel capacity. Thus, the channel overflow and become inundation on the land, especially on the road.

The area that often affected by inundation is Pangeran Antasari road where is inside of Kelayan Regional Inundation Handling Unit (SWPG Kelayan). Based on Banjarmasin City Spatial Planning Map in 2018, the area is a business zone. Therefore this area should receive special attention to resolve the problem of inundation. Various places in this area also produce wastewater, which is disposed of through the stormwater drainage channels. If the sewer cannot drain properly and become an inundation, it
produces an unpleasant odor. This situation very disturbing for the surrounding environment, especially road users itself. Besides, roads that are often flooded by water will also potentially be damaged.

Based on the problems that have been described, this research was carried out to study and provide recommendations for handling the issue. This research evaluated the capacity of the existing drainage channel to drain the stormwater runoff. If the channel overflow, then the channel normalized by increasing the dimension and the slope as needed to reduce the inundation.

2. Reasearch area
The study area (figure 1) covers a total area of 0.35 km$^2$ and is located inside of the Kelayan RegionalInundation Handling Unit in Banjarmasin, South Kalimantan, Indonesia (114°35'35"E-114°36'12"Eand 3°19'27,5"S-3°20'42,5"S). This area is primarily used for the business area and densely residentialarea, with a small area use for local cemeteries. In the existing drainage system, there are 6 primarychannels and 8 secondary channels, which are the channels not enough to cover the entire study area. Therefore, to simplify the analysis, an ideal drainage plan channel network was added in the study area (figure 2). Overall analysis in this study using the existing drainage network system combined with the new design drainage network system (figure 2).
3. Methodology
The study of inundation handling in the urban drainage system is divided into two primary phases. The first phase is hydrological analysis, which is calculating the amount of direct runoff due to rain using the rational method. The second part is the hydraulic analysis, which is modeling the drainage system (figure 2) using the HEC-RAS, evaluating the drainage channel capacity.

3.1 Hydrological Analysis
3.1.1 Rational Method. Hydrological analysis of small catchment can be used as the rational method. A catchment area can be categorized as a small catchment with values ranging from 1.3 to 2.5 km$^2$, but there is no theoretical lower limit [2]. It has particular application in urban storm drainage, where it is used to calculate peak runoff rates for the design of storm sewer or small drainage structures. The formula for the rational method (1) is the following:

$$ Q_p = 0.278 \times C \times I \times A $$

where $Q_p$ is peak discharge corresponding to given rainfall intensity, duration, and frequency (m$^3$/s), $C$ is runoff coefficient ($0 \leq C \leq 1$), $I$ is rainfall intensity (mm/hour), and $A$ is catchment area (km$^2$).

The rainfall data used is from Sungai Tabuk Rain Station for 24 years long, from 1978 until 2011 [3].

3.1.2 Runoff coefficient. In Appendix I of Ministry of Public Works Regulation No.12/PRT/M/2014 about Implementation of Urban Drainage System, specify a table of runoff coefficient values (table 1) in urban areas for various conditions, referring to Design and Construction of Sanitary and Storm Sewers, ASCE Manual of Engineering Practice no.37 (1960).

**Table 1. Runoff coefficient values $C$ for the study area**

| Land use / Type of surface       | $C_i$ |
|----------------------------------|-------|
| Business area (city)             | 0.825 |
| Residential (city), roof         | 0.85  |
| Concrete pavement                | 0.875 |
| Cemetery                         | 0.2   |
| Unused bare land                 | 0.3   |

In urban areas, the catchment area usually consists of different surface characteristics [4]. Therefore a composite analysis is required to calculate the runoff coefficient, it is calculated as follows [5]:

$$ C = \frac{\sum_{i=1}^{n} C_i A_i}{\sum_{i=1}^{n} A_i} $$

where $A_i$ is catchment area with specific land-use type, $C_i$ is the runoff coefficient of specific land-use type, and $n$ is the total amount of land-use types.

3.1.3 Rainfall intensity. The Mononobe method is used to obtain the Intensity-Duration-Frequency (IDF) curve if the data available only daily rainfall data. The Mononobe equation is calculated as follows [6]:

$$ I_t = \frac{R_{24}}{24} \left( \frac{24}{t} \right)^{\frac{2}{3}} $$

where $I_t$ is the rainfall intensity for the duration of rain $t$ (mm/hour), $t$ is the duration of rainfall (hours), and $R_{24}$ is the maximum rainfall for 24 hours (mm).
3.1.4 Time of concentration.

The Kirpich formula is used in this study to obtain of time required for runoff to travel from one control point to another. It is calculated as follows [7]:

\[ t_c = 0.0195L^{0.77}S^{-0.385} \]  

(4)

where \( t_c \) is the time of concentration (minutes), \( L \) is the length of the channel from upstream to the control point (meters), and \( S \) is the average slope of the channel.

3.2 Hydraulics Analysis

3.2.1 HEC-RAS 4.1.0, a platform for constructing the hydraulic analysis of drainage channel system.

The HEC-RAS version 4.1.0 is a hydraulics simulation model based on momentum, mass, and energy conservation laws. In this study, performing one-dimensional water surface calculation for steady gradually varied flow in constructed channels. The water surface profile is computed from one cross-section to the next by solving the energy equations (5) with an iterative procedure called the standard step method. Meanwhile, the momentum equation (6) is used when the water surface profile changes rapidly [8].

The formula of energy and momentum equation is the following:

\[
\begin{align*}
Y_1 + Z_1 + \frac{\alpha_1 V_1^2}{2g} &= Y_2 + Z_2 + \frac{\alpha_2 V_2^2}{2g} + h_f \\
\frac{\partial Q}{\partial t} + \frac{\partial QV}{\partial x} + gA \left( \frac{\partial Z}{\partial x} + S_f \right) &= 0
\end{align*}
\]

(5)  

(6)

where \( Y \) is depth of water at cross-sections, \( Z \) is elevation of the main channel invert, \( V \) is average velocities, \( \alpha \) is velocity weighting coefficients, \( g \) is gravitational acceleration, \( h_f \) is energy head loss, \( x \) is distance measured as flow direction, \( z \) is elevation of water surface, \( t \) is time, and \( S_f \) is slope of the energy line.

The energy head loss is comprised of friction losses and contraction and expansion losses. The equation for the energy head loss is as follows:

\[ h_f = L \bar{S}_f + C \left| \frac{\alpha_2 V_2^2}{2g} - \frac{\alpha_1 V_1^2}{2g} \right| \]  

(7)

where \( L \) is discharge weighted reach length, \( S_f \) is representative friction slope between two cross-section, and \( C \) is the coefficient of energy loss due to changes in cross-section (contraction/expansion). The distance weighted reach length is calculated as:

\[ L = \frac{L_{lob}Q_{lob} + L_{ch}Q_{ch} + L_{rob}Q_{rob}}{Q_{rob} + Q_{ch} + Q_{lob}} \]  

(8)

where \( L_{lob}, L_{ch}, L_{rob} \) is cross-section reach lengths specified for flow in the left overbank, main channel, and right overbank, respectively, and \( Q_{lob}, Q_{ch}, Q_{rob} \) is arithmetic average of the flows between sections for the left overbank, main channel, and right overbank, respectively.

The determination of conveyance is calculated from the following form of Manning’s equation:

\[ Q = K \bar{S}_f^{1/2} \]  

(9)

\[ K = \frac{1}{n} \bar{A} R^{2/3} \]  

(10)

where \( K \) is the carrying capacity of the cross-section, \( n \) is Manning’s roughness coefficient, \( A \) is flow area of the cross-section, \( R \) is the hydraulic radius of the cross-section of each section (area/wetted perimeter).

Existing drainage channel system data of SWPG Kelayan was obtained from the Ministry of Public Works in Banjarmasin City in the form of drainage channel schemes in a figure, channel geometry in a spreadsheet, and topographic data to determine channel elevation in AutoCAD format file.
4. Result and discussion

4.1 Peak runoff discharge

In this study, runoff discharges are calculated using the rational method. The study area is divided into several catchment areas, where each area has one drainage channel. The total discharge of the drainage channel is obtained by accumulating the amount of discharge from the catchment area itself and the previous channel (figure 3). The obtained discharge then used to determine the channel dimensions needed to flow the maximum runoff discharge.

![Figure 3. Peak runoff discharge values for each drainage channel in m³/s](image)

4.2 Evaluation of existing channel capacity combined with the new design drainage network system

The hydraulic analysis was performed using a one-dimensional simulation model by HEC-RAS 4.1.0, with a steady flow simulation. The drainage channel geometry is made based on the existing dimension data and combined with topographic data in the study area to determine the elevation of the surface ground and the channel bottom. For the boundary condition, the maximum discharge from the rational method results is used as a steady flow input data. It also added boundary conditions for the left and right banks of the embankment in the cross-section of the channel. On one side of the channel expanded by using a half road width with a slope of 2%. While for the other side, which is bordered by a residential area or a business area, has assumed expanded as 2 m width. The results of the steady flow simulation show that from 36 existing and new design drainage channels, almost all of the channel has overflow (figure 4), it could not flow runoff discharge due to rain. Thus it becomes inundation on the land also on the road. The maximum inundation height above the levee reaches up to 0.77 m. As an example, the long and cross-section of the channel can be seen in figure 5.
Figure 4. Location of existing and new design drainage channels that overflow

(a) Long section of channel number 4.C.14a - 4.C.14c

(b) Cross-section of 4.C.14c

(c) Cross-section of 4.C.14b

(d) Cross-section of 4.C.14a

Figure 5. Simulation result for the existing channel number 4.C.14a - 4.C.14c

For the velocity in the drainage channel, it following the regulation [7], which is the minimum velocity is 0.6 m/s to prevent sedimentation, while the maximum velocity is 3.0 m/s to prevent erosion at the channel bottom. The velocity is very dependent on the slope of the channel and the material of the
channel wall and bottom. The slope of the existing channel, which is a gentle slope, causes the velocity has very small. Only 12.5% qualify the requirement of velocity, as shown in figure 6.

![Figure 6. Velocity value of each channel as a simulation result](image)

Because of the poor performance of the existing channels to drain the required runoff discharge, therefore the channel needs to be normalized by enhancing the dimensions of the existing channels and increasing the slope of the channel bottom.

4.3 Redesign drainage channel system

The second simulation is performed with different existing channel geometries that used needed channel dimensions based on maximum runoff discharge. Whereas the steady flow input and other boundary conditions are still the same as the first simulation. The results after normalized shows that from 36 normalized existing and new design drainage channels, there are 22 channels do not overflow, while another 14 channels still unable to drain runoff discharge (figure 7). Channels that are still overflowing have a maximum inundation height up to 0.28 m. Fig. 8 shows the results of the second simulation after the channel is normalized in the same channel location as figure 6.

![Figure 7. Location of existing and new design drainage channels that overflow after normalized](image)
For velocity after channel normalized, there is a significant change, which is 56.94% the velocity has to qualify the requirement according to the regulation between the range from 0.6 to 3.0 m/s, as shown in figure 9.

5. Conclusions
The inundation can be concluded because of the lack of existing drainage network and channel capacity that cannot cover the whole area and flows the runoff discharge. Therefore, recommendations are given to handling inundation by making an ideal drainage network planning and normalized the channel by enhancing the dimensions of the channel that has been built. Furthermore, several important things can also be concluded in this study as follows:
1. The overflow from the channel has been reduced from all 36 channels overflow, became only 14 channels that overflow from the entire amount of 36 channels. Mainly for existing channels that have been built, after normalized the channel dimensions, all of the channels will be sufficient to accommodate the runoff discharge.

2. The maximum inundation level that occurs on the ground due to overflow from the channel also reduces, became only 0.29 m height, from 0.69 m height before normalized.

3. The velocity that qualifies the requirement range from 0.6 to 3.0 m/s has been increased, from only 12.5% to be 56.94% can be fulfilled the criteria range.

6. References

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