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Analysis of inundation reduction in drainage channels in the coastal area of the City of Palu with environmental insights

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Abstract. Inundations occur and become problems in the City of Palu. Inundations occur due to outdated drainage systems, high levels of sedimentation, rapid urban development and topography that tends to be flat. The offered alternatives are rehabilitation of existing drainage channels, storage pond equipped with an automatic valve door, rainwater harvesting and gully plugs. Rehabilitation is performed by adjusting drainage depth, with consideration of the shallow groundwater in the area. For rainwater harvesting, the capacity of rain barrels varies from 350-1050 litres, and they are equipped with reservoir wells. With combinations of these alternatives, the inundation reduction effectiveness of catchment area (DTA) 1 is 93.5% and 99.6% for DTA2. Comparing the results to gully plugs, the effectiveness of inundation reduction in DTA1 is 81% and in DTA2 is 98.9%. The Budget Plan for rainwater harvesting varies depending on the capacity of rain barrels, ranging from Rp. 1,492,505.00 to Rp. 2,692,505.00. Gully plugs also vary in costs from Rp. 556,000.00 to Rp. 808,000.00, depending on the size of the gully plug. The first alternative is chosen by the combination of drainage rehabilitation, construction of a storage pond equipped with an automatic valve door, and rainwater harvesting. This is because the reduction effectiveness is greater and the rainwater harvesting maintenance is easier.

Keywords: inundation, reduction, drainage, rainwater harvesting, gully plug.

1. Introduction

As a coastal and developing city, the City of Palu as the capital city of the Province of Central Sulawesi still has many problems. Rapid population growth and rapid economic development have led to increased demand for land. Along with the development of the city area, the area of land and rivers that serve as infiltration areas or reservoirs has decreased. This results in occurrences of inundations at several points in the City of Palu. This study takes the location of East Palu Sub-District, with its flat topography. This area is composed of offices, trade facilities, and dense residential areas. The distribution of flow areas and flow coefficients for each channel is carried out with the help of flow direction maps. The location of the study, as presented in Figure 1, is divided into 2 Catchment Areas (DTA) that are located on the Palu River. Based on the description of the problem, the following is the formulation of the problem for discussion:

- What is the condition of the existing drainage network system on the coast of the City of Palu?
• How is the management of drainage environmentally sound and in accordance with the conditions of the coast of the City of Palu?
• How can inundation management be increased in effectiveness to reduce inundation on the coast of the City of Palu?

2. Materials and Methods
The study location is on the coast of the City of Palu, specifically the Besusu Barat Village and Besusu Tengah Village, East Palu Sub-District, Province of Central Sulawesi. The area is divided into two catchment areas (DTA1 and DTA 2), with outlets on the Palu River.

2.1. Stages of Analysis
Data analysis is performed using existing statistical models and empirical models, by:
1. Hydrological analysis
   • Conducting a consistency test of rainfall data and a periodic test of hydrological data in order to determine whether or not there are data errors or irregularities.
   • Calculating the planned rainfall using the Log Pearson Type III method.
2. Analysis of the drainage system to determine the causes of changes in inundation.
   • Calculating rain intensity with the Mononobe formula.
   • Determining the number of years before inundation returns.
   • Determining the flow coefficient and the width of each drainage channel.
   • Calculating rainwater discharge with modified rational equations.
   • Calculating the discharge of wastewater or waste from the people.
   • Calculating the total discharge or planned flood discharge.
   • Calculating the capacity of existing drainage channels.
   • Calculating the difference between the capacities of the existing drainage channels with the planned discharge to find out if the channels are unable to accommodate the planned discharge.
3. Calculation of the tidal effect on the main channel.
4. Calculation of the backwater effect on the channel outlets with the direct step method.
5. Planning of alternative countermeasures to respond to inundation.

2.1.1. Rainfall Intensity
In general, drainage planning in residential areas, campuses, and city parks utilize a 5-year return period. A higher return means a greater planned discharge. If hourly rainfall data is not available, then the hourly rainfall distribution pattern can be found using the distribution approach and hourly rain ratio using the Mononobe Formula below:

\[ I = \frac{R_{24}}{24} \left( \frac{24}{T_c} \right)^{2/3} \]  

Where:
I = Average rainfall intensity at hour t (mm / hour)
R_{24} = Maximum rainfall in one day (mm / hour)
T_c = Time of rain concentration (hours), for Indonesia 5-7 hours
n = Fixed (estimate for Indonesia: n \sim 2/3)

2.1.2. Discharge Due to Rainfall
The rational method is used to calculate the flood drainage discharge in the form of peak flood discharge so that it includes a non-hydrographic planned flood. The following is the rational formula that has been modified by including the coefficient of shelter (Suhardjono, 2015):

\[ Q = 0,278 \times Cs.C.I.A \]  

Where:
Q = Planned flood discharge (m³ / sec)
C = Runoff coefficient  
I = Rainfall intensity at the duration equal to the concentration time and at a certain return period (mm / hour)  
A = Catchment area (km²)  
Cs = Coefficient of resistance due to storage  
0.278 = Conversion factor (unit order of m³/s)

2.1.3. Backwater Flow
Backwater occurs if the flow is impeded due to buildings or obstacles on the channel. The direct step method is an easy and simple method for calculating water level profiles on non-permanent flows. This method is developed from the energy equation below (Suripin, 2004):

\[ z_1 + h_1 + \frac{V_1^2}{2g} = z_2 + h_2 + \frac{V_2^2}{2g} + h_f \]  \( (3) \)

Where:
- \( z \) = channel base height from the reference line (m)
- \( h \) = water depth from the channel bottom (m)
- \( V \) = average velocity (m/s)
- \( g \) = gravity acceleration (m/s²)
- \( h_f \) = loss of energy due to friction at the base of the channel

3. Results and Discussion

3.1. Catchment Areas (DTA)
The distribution of flow areas and flow coefficients for each channel is carried out with the aid of flow direction maps. The study location is divided into two catchment areas (DTA 1 and DTA 2).

![Figure 1. Distribution of catchment areas (DTA) of the research location](image)

3.2. The Capacity of the Existing Drainage Channels
The results of comparing the capacities of existing drainage channels with the planned flood discharge with return periods of 1.01, 1.5, 2, and 5 years can be seen in Table 1.

1. DTA 1: for a return period of 1.01 years, 4 channels are safe and 7 channels overflow. For a 1.5-year return period, 3 channels are safe and 8 channels overflow. In the condition of a planned 2-year return period, 3 channels are safe and 8 channels overflow. In the condition of a planned discharge at a 5-year return period, 3 channels are safe and 8 channels overflow.
2. DTA 2: for a 1.01-year return period, 64 channels are safe and 3 channels overflow. For a 1.5-year return period, 58 channels are safe and 9 channels overflow. In the condition of a 2-year return period, 55 channels are safe and 12 channels overflow. In the condition of a planned 5-year
return period, 48 channels are safe and 19 channels overflow.

**Table 1.** Comparison of Capacities of Existing Channels and Flood Discharge Plans Based on Return Periods

<table>
<thead>
<tr>
<th>Channel</th>
<th>Existing Channel Capacity (m³/sec)</th>
<th>1.5 Years (m³/sec)</th>
<th>2.0 Years (m³/sec)</th>
<th>5.0 Years (m³/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SR 3 Kn</td>
<td>0.0843</td>
<td>0.0556</td>
<td>0.0644</td>
<td>0.0798</td>
</tr>
<tr>
<td>IHH 1 Kn</td>
<td>0.0833</td>
<td>0.2940</td>
<td>0.3401</td>
<td>0.4215</td>
</tr>
<tr>
<td>SRLP 2 Kr</td>
<td>0.0222</td>
<td>0.3229</td>
<td>0.3736</td>
<td>0.4630</td>
</tr>
<tr>
<td>SH 3 Kr</td>
<td>0.0583</td>
<td>0.1287</td>
<td>0.1489</td>
<td>0.1844</td>
</tr>
<tr>
<td>K 1 Kn</td>
<td>0.1995</td>
<td>0.1048</td>
<td>0.1212</td>
<td>0.1502</td>
</tr>
<tr>
<td>SWJ 1 Kn</td>
<td>0.0467</td>
<td>0.0392</td>
<td>0.0454</td>
<td>0.0562</td>
</tr>
<tr>
<td>HN 1 Kn</td>
<td>0.0400</td>
<td>0.0486</td>
<td>0.0563</td>
<td>0.0697</td>
</tr>
<tr>
<td>HW 4 Kn-r</td>
<td>0.1435</td>
<td>0.0729</td>
<td>0.0843</td>
<td>0.2720</td>
</tr>
<tr>
<td>HW 5 Kr</td>
<td>0.2415</td>
<td>0.0728</td>
<td>0.0842</td>
<td>0.1043</td>
</tr>
<tr>
<td>HW 6 Kr</td>
<td>0.0477</td>
<td>0.0689</td>
<td>0.0797</td>
<td>0.0986</td>
</tr>
<tr>
<td>RJM 5 Kn</td>
<td>0.0738</td>
<td>0.0968</td>
<td>0.1120</td>
<td>0.1387</td>
</tr>
</tbody>
</table>

3.3. **Tidal Analysis**

Based on the results of the tidal analysis using the least squares method, it can be concluded that the type of tides at the study location includes Tidal Mixed Leach to Double Daily (Mixed Tide, Semidiurnal Prevailing) with the highest tide elevation being 2.74 meters. Then, the water level profile was analysed using the HEC-RAS program to determine the effect of tides on the river and channel outlets.

![Figure 2. Cross section of SP2 without tidal effects](image)

![Figure 3. Cross section of SP2 with tidal effects](image)

The backwater analysis was carried out using the Direct Step Method on the main channel (Palu River), where the tidal effect with $h_n = 1.046$ meters had an effect of up to 124.85 meters to the upstream of the channel. Therefore, the backwater effect at the time of the highest tide only affects up to SP3 and DTA 2 outlets. Then, the following backwater calculation due to tidal influences was only performed on DTA 2 outlet. Using the same method, it was found that in DTA 2, the backwater effect on the channel affects up to $+328,291$ meters in the downstream direction, and as high as 0.516 meters in the upstream direction of the channel.

3.4. **Inundation Handling Plan**

The inundation handling plan proposed in this study is the handling of inundation in urban drainage networks that is carried out in an integrated manner and leads to water conservation in both the dry and rainy seasons with the optimization of drainage channels and combination of certain methods. Constraints on the limited availability of land in several locations do not allow the widening of the channel and the creation of new tertiary channels. Therefore the proposed handling includes the following:

1. Rehabilitation of existing drainage channels
2. A storage pond with an automatic valve door
3. Rainwater harvesting
4. Gully plug

In addition, it is also necessary to increase participation from the people as a non-technical factor in the management of integrated and environmentally-friendly drainage channels.

3.4.1 Rehabilitation of Existing Drainage Channels

Efforts to rehabilitate inundation are carried out by changing or improving the dimensions of existing drainage channels based on the results of evaluating the capacity of existing drainage channels. Improvement of the channel dimension is only limited to increasing the depth of the channel because at the study location, it is not possible to widen the channels due to limited land availability. From the analysis, it was found that of the channels in DTA 1, there were only 5 channels that could be rehabilitated out of a total of 8 channels that overflowed. Whereas, in DTA 2, out of a total of 19 channels that overflowed, the channels that can be rehabilitated number to 17. The results of the analysis are shown in Table 2.

<table>
<thead>
<tr>
<th>Channels</th>
<th>Capacity of Existing Channels (m³/sec)</th>
<th>Capacity of Rehabilitation Channels (m³/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SWJ 1 Kn</td>
<td>0.0467</td>
<td>0.0797</td>
</tr>
<tr>
<td>HN 1 Kn</td>
<td>0.0400</td>
<td>0.0768</td>
</tr>
<tr>
<td>RJM 5 Kn</td>
<td>0.0738</td>
<td>0.1600</td>
</tr>
<tr>
<td>YJ 1 Kn</td>
<td>DTA 2</td>
<td></td>
</tr>
<tr>
<td>TD 1 Kn</td>
<td>0.0989</td>
<td>0.2187</td>
</tr>
<tr>
<td>SP 2 Kr</td>
<td>0.2252</td>
<td>0.6247</td>
</tr>
<tr>
<td></td>
<td>0.1853</td>
<td>0.3211</td>
</tr>
</tbody>
</table>

3.4.2 Storage Pond

A storage pond has the function to hold water at a certain time when water cannot flow directly into the river body when the river is in a flooding or tidal state. Besides being used as temporary storage, the storage pond can also be used for other purposes, including water conservation. The storage pond volume is planned to be released from the 4.58-hectare RJM 3 Kr drainage area. From the calculation results, the volume of incoming flow in the storage pond is 311.74 m³, and thus these are the required dimensions of the storage pond:

- Broad (B) = 15.00 metre
- Height (H) = 2.00 metre
- Long (L) = 15.00 metre

This storage pond has a storage capacity of 450 m³.
To find out the safety of the storage pond, the stability of the shear force and rolling style was examined. From the results of calculations, the building was declared safe from the dangers of sliding and rolling. The automatic valve door in the drainage canal in the tidal area functions to prevent the entry of excess water at high tide and to release excess water at low tide. To facilitate operation, the opening and closing of the valve door is performed automatically through utilizing water pressure on the valve. At high tide, pressure on the valve will keep the valve door closed, while at low tide, the valve will be released and the sluice will open. The automatic valve door is made of corrosion-resistant fiberglass, and the planned dimensions of the automatic valve door, based on the calculation results, are $h = 1.4$ meters and $B = 1.4$ meters. The door can start moving if there is a difference in the minimum height of 2-8 cm, based on the provisions of the specification of the PUSAIR PA-FGI type automatic valve door.

3.4.3 Rainwater Harvesting

Rainwater harvesting is the technique of collecting rainwater in a storage tank, natural reservoir, or subsurface aquifers through the absorption of surface water. Rainwater that falls on the surface of roofs can be used for daily needs by accommodating it first; afterwards, the water is made to flow into the reservoir wells. The utilized barrel volumes range from 350 litres to 1050 litres, depending on the intensity of the rain; this is also needed for calculating the limit of the number of storage wells installed in each house (one reservoir well). The effectiveness of inundation reduction by using rainwater harvesting is 93.5% for DTA 1, 99.6% for DTA 2, and 96.5% for both DTA overall. The diagram of rainwater harvesting can be seen in Figure 5 below.

<table>
<thead>
<tr>
<th>No.</th>
<th>Channel</th>
<th>$V_{ab}$ (m$^3$)</th>
<th>Well Dimension</th>
<th>Rain Barrel Volume (m$^3$)</th>
<th>No. of Wells (n)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>IHH 1 Kn</td>
<td>12.7854</td>
<td>L = 1, H = 1.5</td>
<td>10.50</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>SRLP 2 Kn</td>
<td>10.9416</td>
<td>L = 1, H = 1.5</td>
<td>7.50</td>
<td>1</td>
</tr>
<tr>
<td>3</td>
<td>SH 3 Kr</td>
<td>6.8973</td>
<td>L = 1, H = 1.5</td>
<td>3.50</td>
<td>1</td>
</tr>
<tr>
<td>4</td>
<td>SH 2 Kr</td>
<td>10.5529</td>
<td>L = 1, H = 1.5</td>
<td>7.50</td>
<td>1</td>
</tr>
<tr>
<td>5</td>
<td>UDT 1 Kr</td>
<td>8.7749</td>
<td>L = 1, H = 1.5</td>
<td>6.50</td>
<td>1</td>
</tr>
</tbody>
</table>
Figure 5. Rainwater harvesting facility

A single rainwater harvesting system is assumed to be created for each house. Rain barrel volumes vary from 350 to 1050 litres, and the choice depends on the intensity of rain, which is needed to limit the number of reservoir wells (one reservoir well). The dimensions of the well are 1.5 metres in depth and 1 metre in width. The planned unit cost for rainwater harvesting per unit varies from Rp. 1,492,500 up to Rp. 2,692,500, depending on the volume of the rain barrel to be used.

3.4.4 Gully Plug

A gully plug is an infiltration well that functions to absorb water through trenches that are given water reservoirs. Chirping trenches are also infiltration wells that are collective with shallow infiltration wells. Construction of a chute trench, in general, is not much different from an infiltration well, which distinguishes it only from the upper cover. The top cover of an infiltration well utilizes reinforced concrete plates, while a gully plug uses a filter iron plate that is intended to make the water on the channel flow directly into the gully plug.

Planning a gully plug can accommodate water with volumes ranging from 0.0078 m³/s to 0.0153 m³/s, depending on the width and number of planned trenches. The effectiveness of inundation reduction by using gully plug is 81% for DTA 1 and 98.9% for DTA 2, with the overall being 89.9% for both DTA. The diagram of a gully plug can be seen in Figure 6 below.

Table 4. Gully Plug Calculations

<table>
<thead>
<tr>
<th>No.</th>
<th>Channel</th>
<th>Channel Length (m)</th>
<th>Dimension Plan</th>
<th>No. of Gully Plugs (n)</th>
<th>Total Vol. of Gully Plug (m³)</th>
<th>Total Vol. of Gully Plug (m³/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>IHH 1 Kn</td>
<td>12.7854</td>
<td>1</td>
<td>0.6</td>
<td>1.5</td>
<td>68</td>
</tr>
<tr>
<td>2</td>
<td>SRLP 2 Kn</td>
<td>10.9416</td>
<td>1</td>
<td>0.5</td>
<td>1.5</td>
<td>70</td>
</tr>
<tr>
<td>3</td>
<td>SH 3 Kr</td>
<td>6.8973</td>
<td>1</td>
<td>0.7</td>
<td>1.5</td>
<td>68</td>
</tr>
<tr>
<td>4</td>
<td>SH 2 Kr</td>
<td>10.5529</td>
<td>1</td>
<td>0.7</td>
<td>1.5</td>
<td>68</td>
</tr>
<tr>
<td>5</td>
<td>UDT 1 Kr</td>
<td>8.7749</td>
<td>1</td>
<td>0.7</td>
<td>1.5</td>
<td>82</td>
</tr>
</tbody>
</table>
The planned gully plug construction is adjusted to the width of the existing channel, with a depth of $h = 1.00$ meters per ditch and the distance between gully plugs being 5.00 meters. The price of each gully plug varies from Rp. 556,000 to Rp. 808,000 depending on the width of each planned gully plug.

4. Conclusion

Based on the results of the inundation reduction analysis that has been conducted on drainage channels in a coastal area of the City of Palu with environmental insights, the following conclusions are obtained:

- The condition of the existing drainage channel in some areas are not able to accommodate the 5-year planned flood discharge. For DTA 1, the number of existing channels that can accommodate the 5-year flood discharge is 3 channels, while 8 channels overflow. For DTA 2, the number of channels that can accommodate the 5-year flood discharge is 48 channels, while 19 channels overflow. This is caused by the lack of capacity of existing drainage channels and the presence of sedimentation, waste, and, vegetation, as well as the presence of backwater influences on channel outlets in DTA 2.

- The form of environmentally sound drainage management that can be applied to each catchment area is adjusted to the conditions of each area. In some parts, channel rehabilitation is carried out, considering that there is available land that allows for dredging, and that the volume is not too large. Therefore the proposed alternative treatment includes a storage pond equipped with a door with an automatic valve, rainwater harvesting, and gully plugs.

- The effectiveness of inundation reduction from alternative combinations used in rainwater harvesting reaches 96.5% in both catchment areas. Whereas, the combination using gully plugs reaches 89.9% in both catchment areas. From the two alternative treatments, the first alternative is chosen with the use of rainwater harvesting, because the effectiveness of reducing inundation in greater. Further, when compared to the gully plugs, the treatment of rainwater harvesting is much easier.

Acknowledgements

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