The Impacts of Pelosika and Ameroro Dams in the Flood Control Performance of Konaweeha River

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ABSTRACT

Konaweeha watershed is the largest watershed in Southeast Sulawesi with Konaweeha River as the main river. The main issues in Konaweeha Watershed is floods that occur caused damage to infrastructure and public facilities, lowering agricultural production, and cause fatalities. One of the government's efforts to cope with the flooding problem in Konaweeha Watershed is planning the construction of multi-purpose dams in the upstream of Konaweeha Watershed that is Pelosika Dam and Ameroso Dam. Necessary to study the flood control performance of the two dams. Analyses were performed with hydrologic-hydraulic modeling using HEC-HMS software (Hydrologic Modelling System) version 4.0 and HEC-RAS (River Analysis System) version 4.1. The design rainfalls that were used as input to the model were 2 year, 5-year, 10-year and 25 year. Scenarios used in this study are: (1) Existing Scenario (2) Pelosika Dam Scenario; (3) Ameroro Dam Scenario; (4) Pelosika and Ameroro Dams Scenario. The results showed the maximum water surface elevation along the downstream of Konaweeha River in Scenario (2) and (4) were almost the same in the 2 and 5 years return period design flood. However, in case of 10 and 25 years return period, the difference of maximum water surface elevation at downstream of Konaweeha River was slightly significant. Furthermore, the damping efficiency of the peak discharge (at Probably Maximum Flood or PMF) was found to be 71.70% and 18.18% for the individual Pelosika Dam and Ameroro Dam respectively. Further discussion suggests the development of Pelosika Dam as the higher priority rather than that of the Ameroro Dam.

Keywords: Konaweeha River, flood control, Pelosika Dam, Ameroro Dam

1 INTRODUCTION

Konaweeha Watershed is the largest watershed in Southeast Sulawesi Province with Konaweeha River as the main river. The Konaweeha Watershed upstream is located in Kolaka Regency and crosses Regency of North Kolaka, East Kolaka, Konawe, South Konawe, Kendari and flows into the east coast of Southeast Sulawesi. The main problem of Konaweeha Watershed is flooding that occurs every year and disrupts the activities of communities living around the river, lowers agricultural production and causes damage to infrastructure and public facilities as well as public property losses. Based on Water Resources Management Pattern of Lasolo-Konaweeha River Basin, the Government will build four dams in the Konaweeha Watershed, two of them will be held in the upstream area of Konaweeha Watershed (before Wawotobi weir) namely Pelosika Dam and Ameroro Dam. To find out the extent to which the constructions of the two dams in the Konaweeha Watershed upstream will have effects on flood-control in the downstream of the dam, an integrated study needs to be conducted to the dam construction plan as an integrated flood control system.

2 RIVER AND FLOOD INNUNDATION

2.1 Flood and River

River is a natural channel and/or human made in the form of water drainage network along with water in it flowing from upstream to downstream, which is restricted by the river border line on the right and the left. River flooding constitutes an increase of water discharge that occurs in water bodies (Chow et al., 1988).

2.2 Flood Control through Dam Development

Technically, flood control can be done in two ways both structural and non- structural. One of the structural flood control is the construction of dams that are made to manage water resources which serves for the supply of raw water, irrigation water, flood control, and/or hydroelectricity. The presence of a reservoir as a water storage can change the pattern of flood hydrograph at river in which the dam is built. These changes include slowing down the arrival time of flooding and reservoir, the greater the reduction of flood hydrograph outflow flowing into the reservoir downstream (Hydrologic Engineering Center, 1994).

3 THEORITICAL BACKGROUND

3.1 Excess Rainfall

Model that used to estimates the excess rainfall is the Soil Conservation Service (SCS) Curve Number (CN) model, using Equation (1) (Hydrologic Engineering Center, 2000).

$$P_{ea} = \frac{(P - 0.2S)^2}{P \times 0.8S} \tag{1}$$

Where P_{ea} is the accumulated excess rainfall at time t, P is the accumulated rainfall depth and S is the potential maximum retention which calculate by using Equation (2).

$$S = \frac{25400}{CN} - 254 \tag{2}$$

Where CN is the catchment curve number values.

3.2 Hydraulic Flood Routing

The basic concept used in hydraulics flood routing is the concept of conservation of mass (equation 3) and conservation of momentum (equation 4).

$$\frac{\partial (A \times V)}{\partial x} + \frac{\partial A}{\partial t} - q = 0 \tag{3}$$

$$\frac{\partial V}{\partial t} + V\frac{\partial V}{\partial x} + g\left(\frac{\partial h}{\partial x} + S_f\right) = 0 \tag{4}$$

with x is the distance along the river, t is time, A is the cross section of the river, V is flow velocity, h is the height above the reference surface, g is gravity acceleration, S_f is energy slope, and q is lateral flow, where $\frac{\partial h}{\partial x} = \frac{\partial y}{\partial x} - S_0$, with y is the depth of the flow and S_0 is the channel bed slope so that equation 4 can be written as equation 5.

$$\frac{\partial V}{\partial t} + V\frac{\partial V}{\partial x} + g.\frac{\partial y}{\partial x} - g\left(S_0 - S_f\right) = 0 \tag{5}$$

4 TECHNICAL APPROACH

4.1 Research Location

This research was conducted in the Konaweeha Watershed, from the upstream of Pelosika Dam plan until Konaweeha River estuary. Location plan of Pelosika Dam are in Konaweeha River, District Asinua, Konawe, while Ameroro Dam will be built on the Ameroro River, District Uepai, Konawe. Figure 1 presents an overview of the research sites.

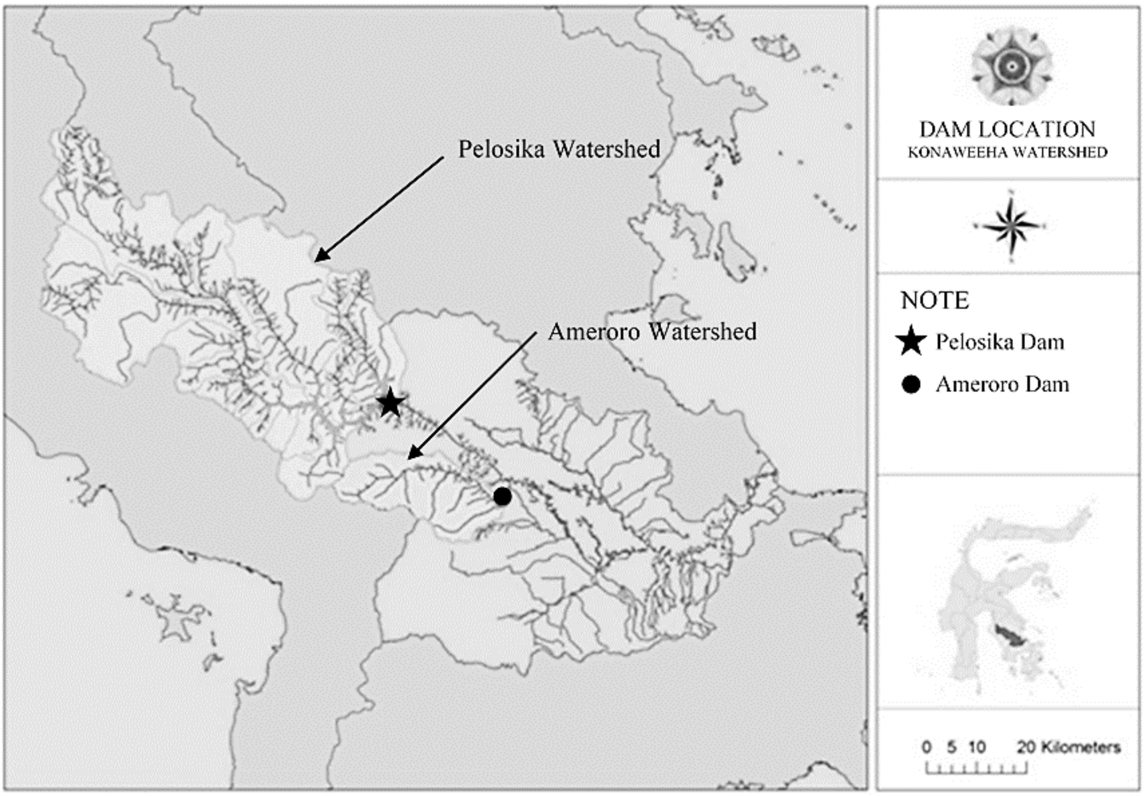

Figure 1. Research sites in Konaweeha Watershed

4.2 Research Approach

In general, the implementation of the study is divided into three main stages covering analysis of hydrology, hydraulics analysis and performance assessment of flood control as described as follows (Sujono, 2014):

a) Hydrological modeling that has been carried out in this study were the analysis of watershed rainfall using the Polygon Thiessen, frequency analysis to determine the amount of design rainfall in the specified return period, pattern of rainfall to determine the distribution of hourly rainfall, the effective rainfall, flood forecasting by the approach of Synthetic Unit Hydrograph (SUH) and watershed system modeling using HEC-HMS software.
b) Hydraulic modeling that has been carried out in this study were the flood routing using HEC-RAS software from the point of confluence of the Konaweeha and the Lahumbuti River.

4.3 Technical Data of Pelosika Dam

The followings describe various technical data being utilized throughout the study implementation (Dian Cipta Dianrancana, 2013).

a) Reservoir elevation
   PMF flood water level : +117.93 mMSL

b) Storage volume
   Dead storage volume : 313.46 MCM
   Active storage volume : 509.10 MCM
   Total storage volume : 822.56 MCM
c) Spillway
   Type of spillway : ogee overflow
   Spillway crest elevation : +113.50 mMSL
   Spillway width : 90 m
d) Dam
   Type of dam : rock fill, central core
   Dam height : 65.00 m
   Dam crest elevation : +119.00 mMSL

4.4 Technical Data of Ameroro Dam

a) Reservoir elevation
   PMF flood water level : +127.29 mMSL
b) Storage volume
   Dead storage volume : 18.86 MCM
   Active storage volume : 31.44 MCM
   Total storage volume : 50.30 MCM
c) Spillway
   Type of spillway : ogee overflow
   Spillway crest elevation : +121.50 mMSL
   Spillway width : 70 m
d) Dam
   Type of dam : rock fill, central core
   Dam height : 68.50 m
   Dam crest elevation : +128.5 mMSL

5 SIMULATION, RESULTS AND DISCUSSIONS

5.1 Design Rainfall

The design rainfall for each catchment are presented in Table 1.

Table 1. Design rainfall for each catchment (mm)

Catchment	Return Period (year)						
	2	5	10	25	50	100	1000
Upstream 1	41.73	51.34	57.77	66.01	72.25	78.60	101.03
Upstream 2	46.68	57.65	64.37	72.41	78.13	83.66	101.32
Ameroro	39.31	51.22	59.48	70.38	78.85	87.62	119.85
Upstream Pelosika	42.96	51.94	57.36	63.76	68.27	72.59	86.24
Downstream Pelosika	54.68	68.35	77.16	88.11	96.19	104.22	131.36
Meraka	68.20	97.51	116.91	141.43	159.62	177.67	237.33
Aopa	38.66	50.95	59.24	69.93	78.06	86.34	115.65
Mowila	40.23	57.57	69.05	83.56	94.32	105.01	140.31
Lahumbuti	39.54	51.45	59.49	69.87	77.78	85.84	114.52
Lahuawu	36.98	52.77	66.29	87.58	107.01	129.99	242.08
Landono	37.25	53.37	66.92	87.92	106.80	128.87	233.48
Boro-boro	54.73	70.74	81.14	94.13	103.74	113.31	145.74
Rambu-rambu	55.30	77.08	91.70	110.34	124.36	138.48	187.18
Downstream	56.83	82.52	100.29	123.46	141.21	159.35	223.55

5.2 Rainfall Distribution

From the observations of rainfall data, a dominant duration is obtained, a 3-hour duration taken to represent ones happening in the Konaweeha Watershed. Rainfall distribution is shown in Table 2.

Table 2. Rainfall distribution of Konaweeha Watershed

Hour	1	2	3
% Rainfall cumulative distribution	41.73	77.53	100
% Rainfall distribution each hour	41.73	35.80	22.47

5.3 Curve Number

Composite CN values for each catchment is presented in Table 3.

Table 3. CN values for each catchment

Catchment	Composite CN	
	CN-II	CN-III
Upstream 1	59.20	76.94
Upstream 2	61.03	78.27
Ameroro	59.21	76.95
Upstream of Pelosika	59.05	76.84
Downstream of Pelosika	63.14	79.76
Upstream of Ameroro	58.89	76.72
Downstream of Ameroro	72.98	86.14
Meraka	72.95	86.12
Aopa	73.19	86.26
Mowila	78.44	89.33
Lahumbuti	72.38	85.77
Ahuawu	75.53	87.65
Landono	74.20	86.86
Arongo	79.65	90.00
Boro-boro	66.99	82.36
Rambu-rambu	80.86	90.67
Alabu	69.69	84.10
Andoroa	85.21	92.98
Ulu-Pohara	53.12	72.27
Polua	55.21	73.92
Mendikonu	54.90	7369
Merataasih	84.37	92.54
Downstream 1	78.14	89.16
Downstream 2	69.21	83.79
Labotoi	78.71	89.48

5.4 Hydrologic and Hydraulic Model Verification

The data of rainfall events used for model verification is the flood events over the period of July 2013 particularly from 15 to 17 July 2013 by referring to the values of the peak discharge during observations on July 16, 2013 namely 1,233 m^3/sec in the Wawotobi Weir. The hydrograph simulation of the flood events in July 2013 by means of Nakayasu and SCS UH approach are presented in Figure 2, in which the Nakayasu provides the result that is closer to the observed data than does the SCS.

For *n*-Manning values calibration, Pohara Bridge located at RS 28058.32 is used, with the estimation of water surface elevation during flood is +5.3 thus the closest obtained value is n = 0.028. Figure 3 presents the results of simulations using n-Manning value of 0.028 at Pohara Bridge.

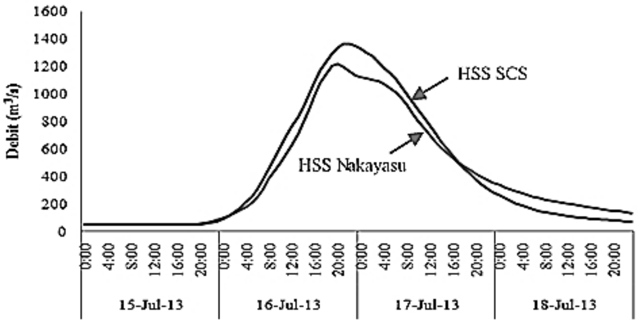

Figure 2. Flood hydrograph July 2013 simulation in Wawotobi Weir

5.5 Dumping efficiency

Figure 4 and 5 present the inflow-outflow hydrograph at the dam spillway using probable maximum flood. Dumping efficiency values are presented in Table 4.

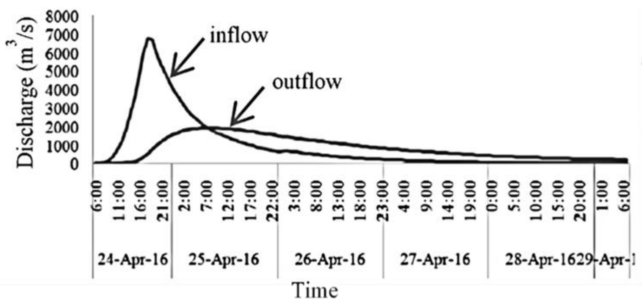

Figure 4. Hydrograph inflow and outflow at the Pelosika dam spillway

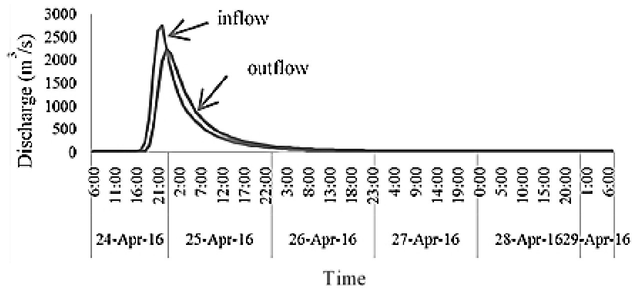

Figure 5. Hydrograph inflow and outflow at the Ameroro dam spillway

Table 4. Dumping efficiency at the dam spillway

Dam name	PMP (mm)	PMF (m^3/s)		
		Inflow	Outflow	DE (%)
Pelosika	214	6,813.22	1,928.19	71.70
Ameroro	300	2,745.76	2,246.50	18.18

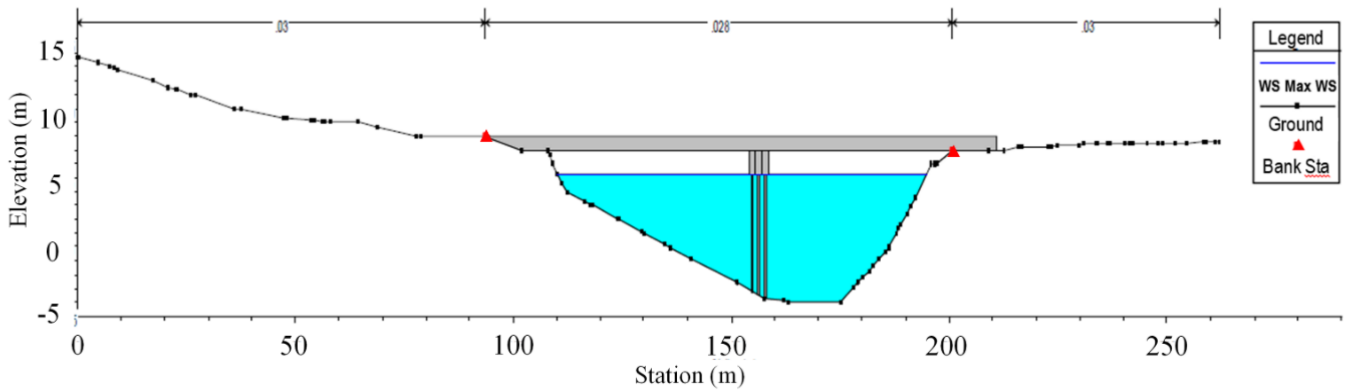

Figure 3. The results of the hydraulic simulation at the Pohara Bridge with n = 0.028

5.6 Flood Discharge

The percentage of reduced flood discharge at the meeting point of the Konaweeha River and the Lahumbuti River is presented in Table 5.

Table 5. Decrease of the maximum flood discharge (m^3/s)

Scenario	Decrease of maximum flood discharge relative to Scenario 1 (%)			
	2 year	5 year	10 year	25 year
Scenario 1 (existing)	0.0	0.0	0.0	0.0
Scenario 2 (Pelosika Dam)	15.7	13.0	26.0	14.0
Scenario 3 (Ameroro Dam)	15.	11.8	2.4	0.4
Scenario 4 (Pelosika and Ameroro Dam)	32.1	22.7	32.0	15.7

From Table 5, it can be seen that the difference between maximum flood discharge reduction between the Scenario 2 (Pelosika Dam scenario) and the Scenario 3 (Ameroro Dam scenario) is not too significant in 2 to 5-year return period. Considerable difference seems to appear at the simulation using the 10- year and 25-year return period rainfall with an assumption that watershed conditions are wet, showing that the Pelosika Dam catchment contributes greatly to reduce floods in the Konaweeha River and therefore the presence of the Pelosika Reservoir will give a greater damping effect on flooding in the Konaweeha River.

5.7 Maximum Water Surface Elevation

The control point to monitor the changes of water surface elevation are presented in Table 6. Table 7 to 10 present the maximum water surface elevation resulted from HEC-RAS simulation (Δ is the difference of maximum water surface elevation at certain locations).

Table 6. River segment that used to monitor the changes of maximum water surface elevation

No	River segment			Control Point (RS)	Location		
					Village	District	Regency
1	30801.33	-	30400	30400	Andepali	Ranomeeto Barat	Konsel
2	28212.98	-	25995.43	26801.95	Pohara	Sampara	Konawe
3	25399.15	-	24388.21	24798.19	Andodawi	Sampara	Konawe
4	23798.76	-	22600	23196.58	Polua	Sampara	Konawe
5	22235.27	-	21404.32	21816.46	Mendikonu	Sampara	Konawe
6	21144.76	-	16405.37	20402.28	Besu	Bondoala	Konawe
7	16165.79	-	13802.79	15554.61	W.Morihi	Bondoala	Konawe
8	13594.14	-	10805.26	13594.14	Laosu	Bondoala	Konawe
9	5402.959	-	3444.525	5402.959	Laosu	Kapoala	Konawe
10	1260.097	-	56.0124	828.9919	Batugong	Kapoala	Konawe

Table 7. Maximum flood water surface elevation for 2 years return period

No	Control	Scenario I		Scenario II		Scenario III		Scenario IV	
	Point (RS)	Elevation (m)	Δ	Elevation (m)	Δ	Elevation (m)	Δ	Elevation (m)	Δ
1	30400	4.21	0	3.92	0.29	3.92	0.29	3.55	0.66
2	26801.95	3.56	0	3.31	0.25	3.31	0.25	2.99	0.57
3	24798.19	3.46	0	3.22	0.24	3.22	0.24	2.91	0.55
4	23196.58	3.38	0	3.14	0.24	3.14	0.24	2.84	0.54
5	21816.46	3.3	0	3.07	0.23	3.07	0.23	2.78	0.52
6	20402.28	3.18	0	2.95	0.23	2.95	0.23	2.67	0.51
7	15554.61	2.73	0	2.53	0.2	2.53	0.2	2.28	0.45
8	13594.14	2.54	0	2.35	0.19	2.34	0.2	2.11	0.43
9	5402.959	1.85	0	1.72	0.13	1.72	0.13	1.56	0.29
10	828.9919	0.93	0	0.92	0.01	0.92	0.01	0.92	0.01

Table 8. Maximum flood water surface elevation for 5 years return period

No	Control	Scenario I		Scenario II		Scenario III		Scenario IV	
	Point (RS)	Elevation (m)	Δ	Elevation (m)	Δ	Elevation (m)	Δ	Elevation (m)	Δ
1	30400	5.11	0	4.81	0.30	4.84	0.27	4.56	0.55
2	26801.95	4.32	0	4.07	0.25	4.1	0.22	3.85	0.47
3	24798.19	4.19	0	3.95	0.24	3.98	0.21	3.74	0.45
4	23196.58	4.08	0	3.85	0.23	3.87	0.21	3.65	0.43
5	21816.46	3.98	0	3.76	0.22	3.78	0.20	3.56	0.42
6	20402.28	3.85	0	3.63	0.22	3.65	0.20	3.43	0.42
7	15554.61	3.32	0	3.12	0.20	3.15	0.17	2.95	0.37
8	13594.14	3.11	0	2.92	0.19	2.94	0.17	2.76	0.35
9	5402.959	2.26	0	2.13	0.13	2.14	0.12	2.01	0.25
10	828.9919	0.96	0	0.95	0.01	0.95	0.01	0.93	0.03

Table 9. Maximum flood water surface elevation for 10 years return period

No	Control	Scenario I		Scenario II		Scenario III		Scenario IV	
	Point (RS)	Elevation (m)	Δ	Elevation (m)	Δ	Elevation (m)	Δ	Elevation (m)	Δ
1	30400	9.28	0	8.26	1.02	9.16	0.12	7.92	1.36
2	26801.95	7.92	0	6.98	0.94	7.82	0.10	6.64	1.28
3	24798.19	7.65	0	6.74	0.91	7.55	0.10	6.41	1.24
4	23196.58	7.31	0	6.48	0.83	7.22	0.09	6.16	1.15
5	21816.46	7.09	0	6.3	0.79	7.01	0.08	6	1.09
6	20402.28	6.87	0	6.13	0.74	6.8	0.07	5.83	1.04
7	15554.61	6.16	0	5.34	0.82	6.07	0.09	5.04	1.12
8	13594.14	5.97	0	5.13	0.84	5.87	0.10	4.88	1.09
9	5402.959	4.55	0	3.83	0.72	4.46	0.09	3.62	0.93
10	828.9919	1.56	0	1.19	0.37	1.51	0.05	1.15	0.41

Table10. Maximum flood water surface elevation for 25 years return period

No	Control	Scenario I		Scenario II		Scenario III		Scenario IV	
	Point (RS)	Elevation (m)	Δ	Elevation (m)	Δ	Elevation (m)	Δ	Elevation (m)	Δ
1	30400	10.22	0	9.45	0.77	10.2	0.02	9.39	0.83
2	26801.95	8.82	0	8.05	0.77	8.81	0.01	8	0.82
3	24798.19	8.52	0	7.77	0.75	8.52	0.00	7.71	0.81
4	23196.58	8.13	0	7.4	0.73	8.13	0.00	7.35	0.78
5	21816.46	7.94	0	7.17	0.77	7.93	0.01	7.13	0.81
6	20402.28	7.76	0	6.92	0.84	7.76	0.00	6.9	0.86
7	15554.61	7.27	0	6.23	1.04	7.26	0.01	6.18	1.09
8	13594.14	7.08	0	6.04	1.04	7.08	0.00	5.99	1.09
9	5402.959	5.46	0	4.6	0.86	5.45	0.01	4.56	0.90
10	828.9919	2.19	0	1.59	0.60	2.09	0.10	1.57	0.62

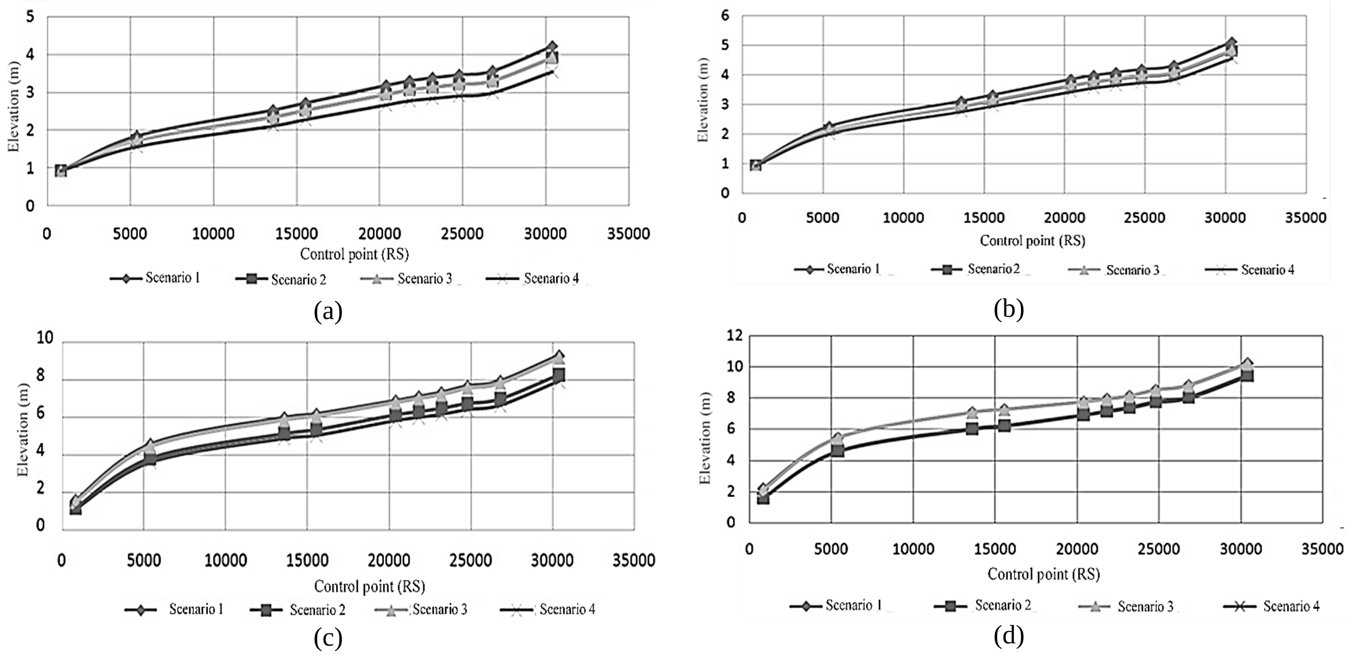

Figure 6. Difference of maximum water surface elevation for elevation for (a) 2 years return period, (b) 5 years return period, 10 years return period, (d) for 25 years return period.

Figures 6 presents results of flood simulations with 4 scenarios. It can be seen that there is no significant difference between the Pelosika Dam scenario and the Ameroro Dam scenario for the 2 to 5-year return period. It shows that both the Ameroro Dam and the Pelosika Dam have almost the same effectiveness in reducing the flood-water surface for 2 to 5-year return period. The differences begins to appear in 10 to 25-year return period (see Table 9 and Table 10), in which it appears that the maximum water level reduction of the Scenario 2 (the Pelosika Dam) ranged from 0.37 meters to 1.02 meters, while the maximum water level reduction of the Scenario 3 (Ameroro Dam) is only 0.12 meters. Furthermore, it also can be seen, for the 10 to 25-year return period design, no significant differences have been found between the Scenario 1 and the scenario 3 and likewise between the Scenario 2 and the scenario 4. The construction of the two dams is more effective in lowering water surface compared to one dam in return period of 2 to 5-year, despite the difference is not so significant.

5.8 Flood Inundation

The recapitulation of number of cross section that spill out from the river bank is presented by Table 11 and Figure 7.It can be seen from the results that for the flood with 2-year return period, the inundation does not occur in all sections of the river. Flood-water begins to appear in the 5-year return period, from Figure 7 there are only 10 cross sections spilling out from the river bank and potentially causing inundation. In the 5-year return period, there is no difference between the scenario 2 and the scenario 3, this can be seen in Table 11 in which the decline level is simply the same namely 58.33%, while the scenario 4 removes all the floodwaters on all cross section reviewed. Moreover, from Table 11, it can be seen that for the 10-year return period, the Pelosika Dam can reduce the number of cross section that spills out from the river bank to 19.35% whereas the Ameroro Dam does not lead to reducing the number of cross section spill out from the river bank. In the 25-year return period, it can be seen that the effectiveness of the dams in reducing the number of inundation points is getting decreased compared to the lower return period.

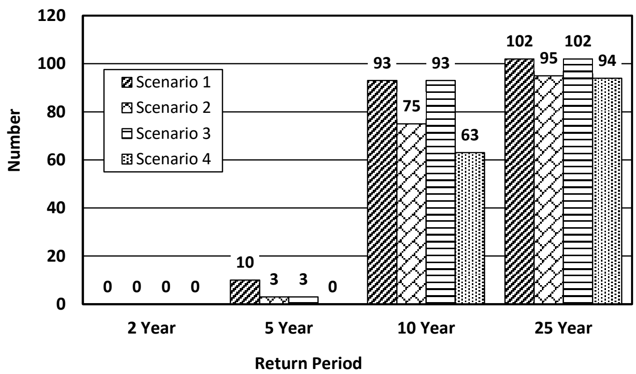

Figure 7. Graph of number of cross section that spill out from river bank

Table 11. Recapitulation of cross section that spill out from river bank

Scenario	Return period											
	2 year			5 year			10 year			25 year		
	∑	Reduction		∑	Reduction		∑	Reduction		∑	Reduction	
		Δ	%		Δ	%		Δ	%		Δ	%
Scenario 1	0	-	-	10	-	-	93	-	-	102	-	-
Scenario 2	0	-	-	3	7	70	75	18	19.35	95	7	6.86
Scenario 3	0	-	-	3	7	70	93	0	0	102	0	0.00
Scenario 4	0	-	-	0	10	10	63	30	32.26	94	8	7.84

6 CONCLUSIONS AND SUGGESTIONS

6.1 Conclusions

From the various analysis that have been conducted in this study, some conclusions are as follows:

a) Damping efficiency based on the inflow and outflow hydrograph at the Pelosika Dam spillway using PMF is 71.70% and Ameroro Dam 18.18%.
b) In the flood simulation for 2 to 5-year return period, there is no significant differences in flood-water level between the Pelosika Dam and the Ameroro Dam. The significant difference between the two dams occurs in 10 to 25-year return period. The Ameroro Dam does not give significant reduction effect compared to the Pelosika Dam.
c) The downstream area of the Konaweeha River as the flood plan area is able to accommodate the flood discharge of the 2-year return period. The construction of the dam may effectively reduce the potential inundation in 5 to 10-year return period, yet it is less effective to 25-year return period flood.

6.2 Suggestions

Some suggestions which can be put forward for the purpose of flood-control in the Konaweeha Watershed are as follows.

a) The efforts in controlling flood can be conducted not only in structural but also in non-structural ways. There is a need for in-depth studies to deal with the flood problem at Konaweeha Watershed which not simply rely on structural ways.
b) The limitation of the hourly data is a major constraint in the making of hydrology and hydraulic models in order to approach the real system, therefore it is necessary to provide a real time hydrological observation post.
c) Since the most potentially affected areas by floods are those in the downstream of Konaweeha Watershed, it is necessary to create an instrument for flood early-warning system in order to minimize the potential loss of both property and human lives.
d) Since there is no significant difference in reducing the water surface elevation at downstream of Konaweeha River, the development of Ameroro Dam may be put at the next priority after the Pelosoka Dam is built.

ACKNOWLEDGEMENTS

Thanks are forwarded to the Ministry of Public Works for the provision of scholarship and facilities and also to the local government of Kolaka Regency, Southeast Province, for the provision of data that make this study is more reliable.

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