湖南大学学报（自然科学版）
Journal of Hunan University (Natural Sciences)

第 51 卷 第 6 期
2 0 2 4 年 6 月

Available online at http://jonuns.com/index.php/journal/index

Vol. 51 No. 6
June 2024

Open Access Article

https://doi.org/10.55463/issn.1674-2974.51.6.10

Flow Characteristics due to End Sill Addition in the Flow along the Spillway Channel

Yogi Pandhu Satriyawan[1]*, Very Dermawan[2]*, Pitojo Tri Juwono[2]

[1] Doctoral Student, Department of Water Resources, Faculty of Engineering, University of Brawijaya, Malang, Indonesia

[2] Department of Water Resources, Faculty of Engineering, University of Brawijaya, Malang, Indonesia

* Corresponding authors: pandhuyogi@gmail.com, peryderma@ub.ac.id

Received: March 8, 2024 / Revised: April 2, 2024 / Accepted: May 10, 2024 / Published: June 28, 2024

Abstract: This research aims to investigate the flow characteristics with the end sill addition at the first and second points in the flow along the spillway channel. The research location is the Bagong Dam in the Bagong River, Trenggalek Regency, East Java Province, Indonesia. Bagong River is in the Ngrowo-Ngasinan Sub-watershed, a sub-watershed of the Brantas Watershed. The Bagong River contributes to heavy floods, so the flood control fully fulfilled the flood requirements in Trenggalek City and surrounding areas. The methodology consists of data collection of daily rainfall, analysis of the design rainfall using frequency analysis, analysis of the hourly rainfall distribution, analysis of the efficient rainfall, and analysis of the flood hydrograph as input to computational fluid dynamics (CFD). The result is a design alternative for determining the two end sill points used to determine the flow pattern and recommend energy dissipators in the Bagong Dam.

Keywords: end sill, spillway, Bagong Dam.

由於沿著溢洪道的水流中添加了端坎而產生的流動特性

摘要：本研究旨在调查在泄洪道沿线流动的第一和第二个点处添加端坎的流动特性。研究地点是印度尼西亚东爪哇省特伦加列克县巴贡河的巴贡大坝。巴贡河位于恩格罗沃 - 恩加西南子流域，该子流域是布兰塔斯流域的一个子流域。巴贡河是洪水泛滥的罪魁祸首，因此防洪措施完全满足了特伦加列克市及其周边地区的防洪要求。该方法包括每日降雨数据收集、使用频率分析分析设计降雨、分析每小时降雨分布、分析有效降雨以及分析洪水过程线作为计算流体动力学(差价合约)的输入。结果是一种设计方案，用于确定用于确定流动模式的两个端坎点，并推荐巴贡大坝中的能量消能器。

关键词：底坎、溢洪道、巴公壩。

1. Introduction

Watershed management is a human effort to arrange the reciprocal relationship between natural resources and humans in the watershed along with all activities, so it has materialized the sustainability and compatibility of ecosystems and increased the sustainable utilization of natural resources for humans. An effort to sustain the ecosystem is the construction of dams. A dam is an infrastructure building holding water resources that can be utilized by the local population. In addition, it is useful for supporting the increase in socio-economic status [1] by fulfilling food self-sufficiency [2], irrigation, conservation efforts [3], hydro-electrical power, flood control, tourism, and others [4]. One of them is the development of the Bagong Dam in Sumurup and Sengon villages, Bendungan district, Trenggalek Regency, East Java province, which was written in the President Rule of Indonesian Republic No 3, 2016 about the implementation acceleration of the national strategy project.

The spillway is functioned for flowing flood water that causes an increase in the water level in the dam. It is important to avoid over-topping [5]. Based on the design of 2023, there are hydraulic jumps downstream of the stilling basin because the condition of the channel to the original river directly hits the river wall, so adding an end sill is a solution, so it does not grind the river wall. Additionally, an end sill in the spillway downstream of the Bagong Dam requires further research on the water flow in the spillway until the original river. The end sill can be added alternatively by simulating the spillway channel as an energy dissipator so that there is no hydraulic jump. The occurrence of a super-critic flow and hydraulic jump becomes the trigger for scouring that can cause building collapse [6]. The operation of the main spillway function adds a safety factor to the discharge capacity of flood discharge under emergency conditions and cannot be operated for reservoir optimization [7].

The Bagong Dam is designed in the Bagong River and is located in the Trenggalek Regency. Bagong River is a river in the Ngrowo Sub-Watershed in Ngasinan and is a sub-watershed in the Brantas Watershed. The Bagong River contributes to many floods, so the flood control of Ngrowo-Ngasinan-Parit Agung-Parit Raya has not completely fulfilled the flood requirements in Trenggalek City and surrounding areas. A dam is a water resource infrastructure with very high risk [8-9]. If it fails, one of them is the development of a spillway, which has a very high risk when it experiences a water jump [7, 10] that can cause the collapse of a building.

The spillway type of the Bagong Dam is overflowing, and the elevation of the spillway is +325 m. The flood discharge of QPMF is 589 m^3/s, and the flood discharge of Qrout PMF is 567 m^3/s. The overflow to the spillway is big enough, with a “damping” pool type of USBR Type III with a width of 30 m and length of 15 m. One of the aspects to be considered in dam design is the hydraulic condition of the spillway [11], so it is necessary to analyze the water flow from the spillway to the original river.

In 2021, a physical model test was conducted in the Laboratory of Applied Hydraulics, Department of Water Resources, Faculty of Engineering, and University of Brawijaya-Malang. The result is as follows: Q1.01 years until Q200 years in launcher channel I, the flow depth has been equal; however, between Q1000 years until QPMF, the flow depth is less equal upstream of launcher channel I. Therefore, this study is an alternative by adding an end sill in several sections of the spillway channel to obtain a uniform flow [12-13]. This research investigates the flow characteristics in addition to the end sill at the first and second points in the flow along the spillway channel.

2. Materials and Methods

2.1. Design Flood

Table 1 shows the discharge values of the inflow and outflow at the spillway based on the Model Test Report (2021).

Table 1 Design floods of inflow and outflow (based on the model test 2021) (The authors)

Return period (year)	Review design 2019		Hd (m)	Elevated water level (m)
	Q_{inflow} (m^3/s)	$Q_{Outflow}$ (m^3/s)		
1.01	78.77	52.86	0.94	325.94
2	131.58	112.66	1.50	326.50
5	163.66	143.81	1.74	326.74
10	182.39	162.18	1.87	326.87
25	203.92	183.04	2.01	327.01
50	218.68	197.60	2.10	327.10
100	236.33	214.76	2.21	327.21
200	245.60	223.96	2.27	327.27
1000	274.26	251.93	2.44	327.44
½ PMF	300.65	277.49	2.58	327.58
PMF	599.48	572.02	4.02	329.02

2.2. Spillway

Each reservoir has a certain water storage capacity. If the water storage experiences full of water, the reservoir surface will increase, and the water will overflow through the dam peak. To avoid this problem, there is a need for a structure so that water can flow downstream [14]. Therefore, a spillway is needed to flow the water. The spillway structure can be separated from the dam body.

Spillway structures have a definition that is a hydraulic structure that is built for flowing floods through a dam without endangering the dam safety. There are many types of spillway structures, which are controlled or uncontrolled. The two types of structures if built can adapt to the field conditions. The controlled spillway channel is completed with a gate that can be raised or lowered in accordance with the reservoir operation pattern during a flood. When the reservoir water level is full of it will be the same as the spillway crest level.

2.3. Flow Capacity through Spillway

Reservoir flood routing is based on the continuity equation as follow [15-16]:

$$\frac{I_1 + I_2}{2}\Delta t + \left(S_f - \frac{Q_1}{2}\Delta t\right) = \left(S_2 + \frac{Q_2}{2}\Delta t\right) \quad (1)$$

or

$$\frac{I_1 + I_2}{2}\Delta t + \left(\frac{S_1}{\Delta t} - \frac{Q_1}{2}\right) = \left(\frac{S_2}{\Delta t} + \frac{Q_2}{2}\right) \quad (2)$$

If

$$\left(\frac{S_1}{\Delta t} - \frac{O_1}{2}\right). = \psi_1 \quad (3)$$

$$\left(\frac{S_2}{\Delta t} - \frac{O_2}{2}\right). = \varphi_2 \quad (4)$$

so,

$$\frac{I_1}{I_2} + \psi_1 = \varphi_2 \quad (5)$$

where I_1 - inflow at the beginning of Dt; I_2 - inflow at the end of Dt; O_1 - outflow at the beginning of Dt; O_2 - outflow at the end of Dt; S_1 - storage at the beginning of Dt; S_2 - storage at the end of Dt; Dt - period of flood routing (3.600 a).

I_1 and I_2 are known from the inflow hydrograph to the reservoir if the routing period has been determined. S_1 is the reservoir storage at the beginning of the routing period, which is measured from the data of the production facility (crest of spillway). However, an overflow-type spillway is one of the components at approach channel that is made for further improving the regulation and enlarging the water discharge through the spillway. Fig. 1 shows the reservoir flood routing through the spillway.

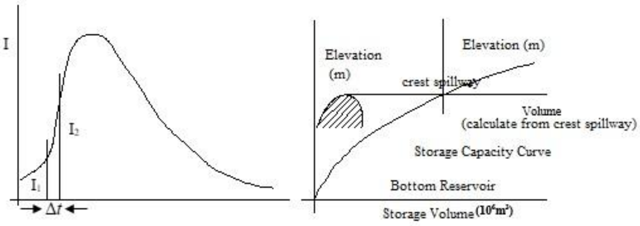


Fig. 1 Reservoir routing through a spillway [4]

To determine the discharge through the spillway, the formulation is [17]:

$$Q = C.\ L.\ H^{3/2} \quad (6)$$

where Q - discharge through the spillway (m^3/s), C - overflow coefficient ($m^{0.5}$/s), L - effective width of the spillway (m), H - total of water pressure head over the spillway (including water velocity head at approach channel) (m).

The overflow coefficient at the spillway, generally between 2.0 and 2.2, is influenced by several routes as follows: water depth in the spillway upstream, slope at the spillway upstream, and water head over the spillway.

2.4. Water Level Profile

2.4.1. Spillway

By using the continuity equation, the analysis of the water level profile at the spillway with Froude: L < 10m can be solved using the formula [11]:

$$V_Z = \sqrt{2g(Z + H_0 - Y_Z})\quad (7)$$

where VZ - velocity at point Z (m/s), YZ - water depth at point Z (m), Z - height of the review point under the spillway elevation (m), H0 - design energy head over the spillway (m), FZ - Froude number at point Z. Fig. 2 shows a sketch of the overflow spillway.

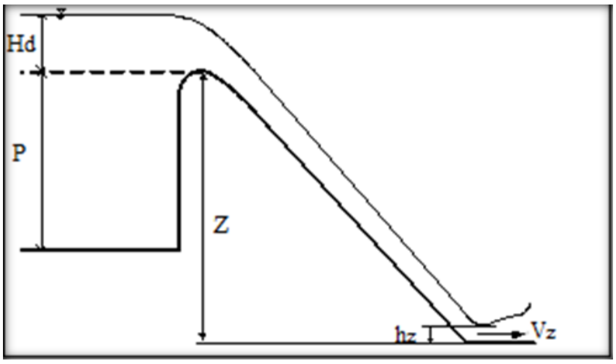


Fig. 2 Sketch of the overflow spillway [2]

2.4.2. Chute Way

In designing the chute way, it must meet the following requirements: 1) The overflow water from the approach channel flows fluently without hydraulic obstacles; 2) The chute way construction is sturdy and stable enough to store all loads; and 3) The construction cost should be as economical as possible.

To meet these requirements, there is needed to attend to the following: 1) The cross section of the chute way as the benchmark is made as rectangular shape; and 2) the channel bed slope works in such a way so that there is not appear the cavitation symptom that breaks the channel bed surface, and the pulsating that is generated will be able to give a negative effect on the stability of the chute way flow. Fig. 3 presents the flow scheme in the chute way.

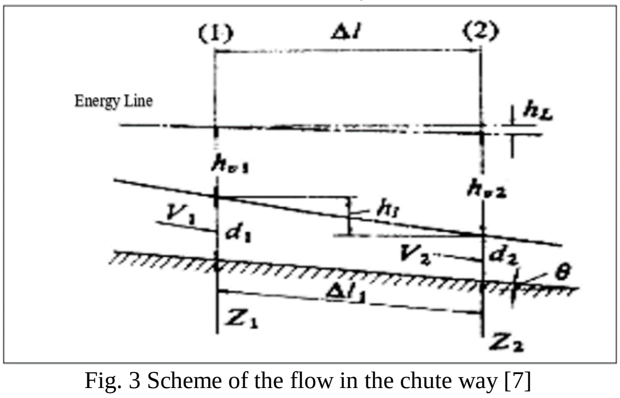


Fig. 3 Scheme of the flow in the chute way [7]

The analysis of water level profile at the chute way can be approached with the formula of energy conservation in flow (Bernoulli formulation) as follows [9, 7]:

$$z1 + d1 + hv1 = z2 + d2 + hv2 + hL + he \quad (8)$$

where z - elevation of the channel bed in a long field (m); d - water depth in the long field (m); hv - velocity head in the long field (m); hL - pressure head loss between two determinant long fields due to pulsating (m), which is expressed with:

$$h_L = Sf.\Delta l \;\; ; \;\; S_f = \frac{\bar{n}^2 \bar{v}^2}{\bar{R}^{4/3}} \quad (9)$$

where he - water head loss due to cross-sectional change (m)

$$h_e = K.\left| \frac{v_1^2 - v_2^2}{2g} \right| \quad (10)$$

where g - gravitational force (m/s^2)

Pulsating flow is a hydraulic phenomenon that must be considered in the chute way structure. If it happens, it will cause uneven flow, so the hydrodynamics power that happened endangers the stability of the construction. In addition, as the reason for the unevenness of the flow, the flow velocity at the chute way foot is uneven decreasing the effectiveness of damping.

In a long chute way, there is an instability danger in the flow that is mentioned as a slug/pulsating flow. If the long channel is more than 30 m, it must be controlled by calculating numbers of Vendernikov (V) and Montuoris (M).

Number of Vendernikov (V) [15]:

$$\underline{V} = \frac{2bv}{3P\sqrt{gd\cos\theta}} \quad (11)$$

Number of Montuoris (M):

$$\underline{M}^2 = \frac{v^2}{gIL\cos\theta} \quad (12)$$

where b - channel bed width (m); v - flow velocity (m/s); g - gravitation (= 9.81 m/s^2); P - wet perimeter (m); d - hydraulic depth (m); I - mean slope of energy gradient (= tan Θ); Θ - angle of energy gradient; L - channel length (m).

The analysis value of the two equations is then plotted in the graphic to determine whether or not the pulsating flow. Fig. 4 presents the pulsating flow criteria.

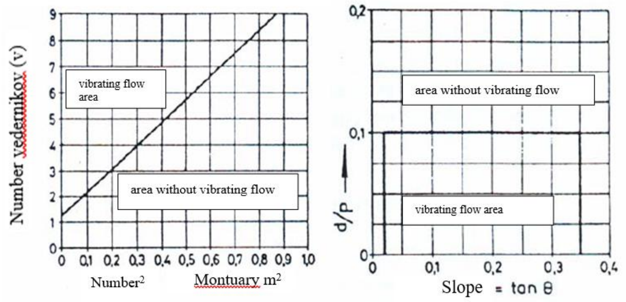


Fig. 4 Pulsating flow criteria (USBR 1978) [10]

2.4.3. Stilling Basin

To muffle the flow after passing the chute way, damping is needed so that the flow enters the approach channel downstream and returns into sub-critical condition. However, the choice of stilling basin type is based on the Froude number. Fig. 5 presents the design of the stilling basin and jump type.

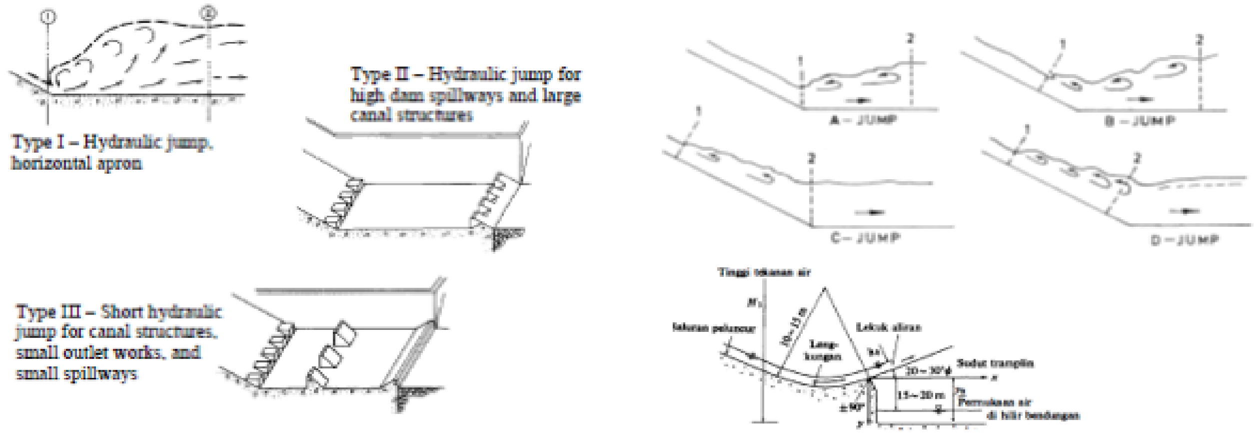


Fig. 5 Design of stilling basin and jump types [11]

2.5. Process of Physical Model

2.5.1. Preparation of the Spillway Structure in a Model Test

The preparation and development of spillway is started on the second until third week. The process of structure building takes about 7-8 weeks. The documentation is presented in Fig. 6.

2.6. Study Location and Methodology

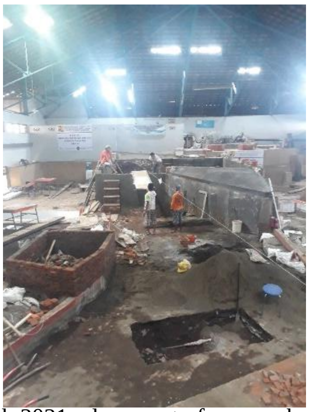
The study site of the Bagong Dam is in the Bagong River and is administratively part of Bendungan District, Trengglalek Regency, East Java Province (Figs. 7 and 8).

The topography is dominant of hills, with slopes ranging from moderate to less steep. The hills are aligned in an elongated river line with depths varying from 100 to 500 m. The river width surrounded design as is between 5–15 m with the river basin is “V” shape and the cliff slope in the right and left is between 15° and 60°. The width of the top side span of the river basin in the as location is between 100–300 m. Fig. 7 presents the map of the study area, and Fig. 8 presents the map of the spillway in the Bagong Dam.

17th March 2021, stacking of main building

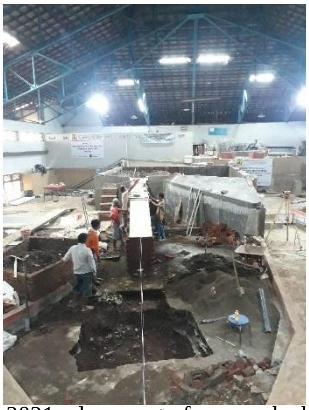
24th March 2021, building stacking is almost complete

26th March 2021, excavation on the location of the stilling basin



29th March 2021, placement of approach channel I

30 March 2021, placement of spillway merck

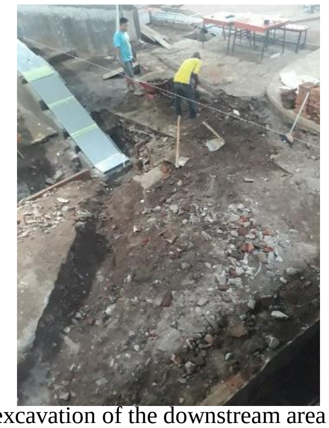


1st April 2021, placement of approach channel II

5th April, 2021, placement of approach channel III

7th April 2021, placement of R at connection of approach channels I–II and II–III

8th April 2021, Installation of the chute way wall



9th April 2021, excavation of the downstream area for river contour

21st April 2021, development of river contours

23rd April 2021, Installation of barrier block at stilling basin

Fig. 6 Preparation of model test (The authors)

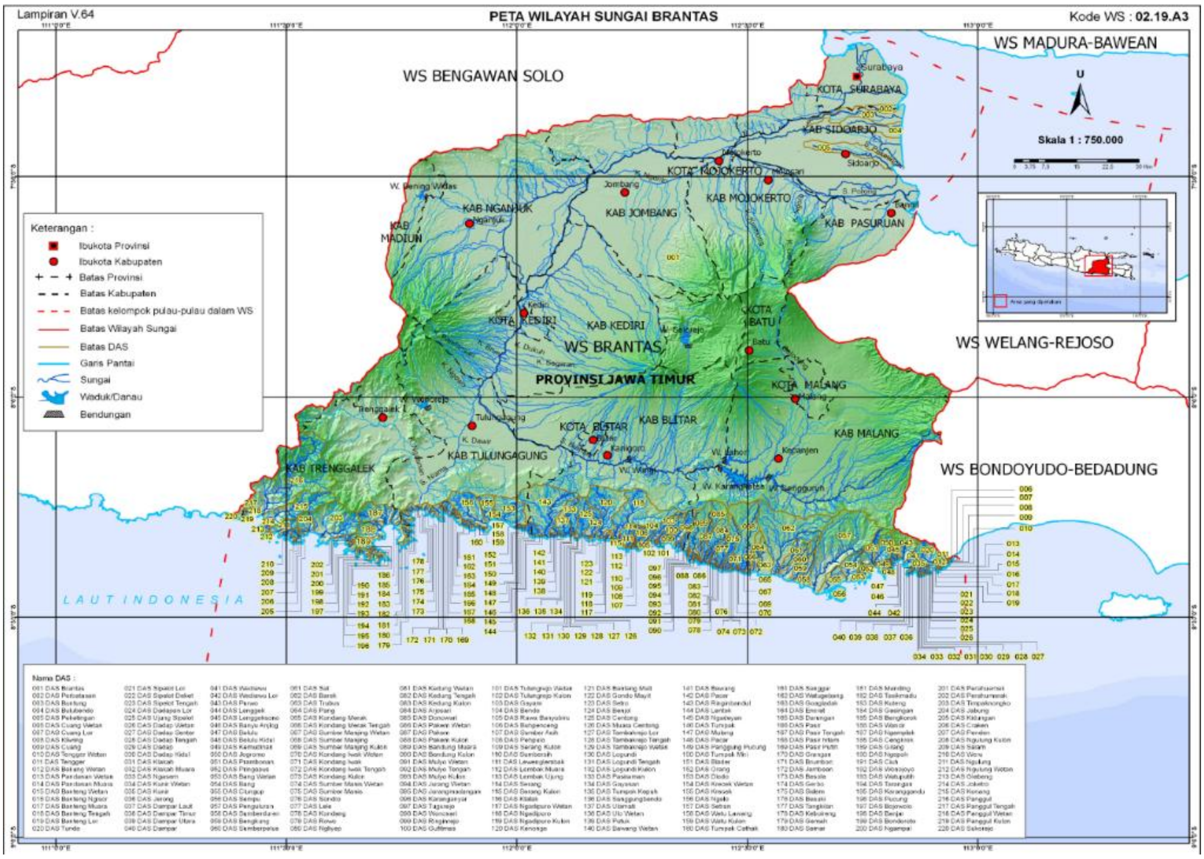


Fig. 7 Map of the Bagong Dam location [19]

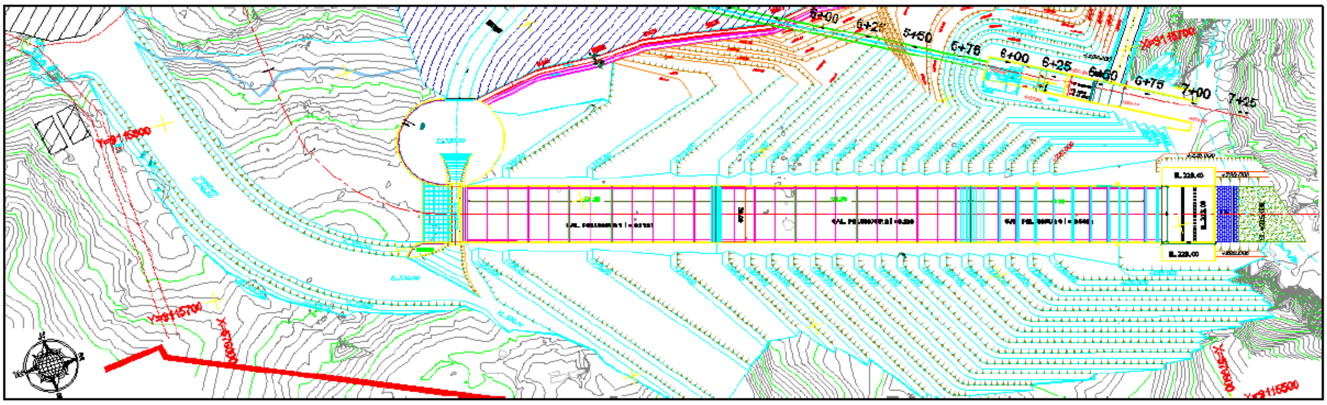


Fig. 8 Spillway map of Bagong Dam (The authors, 2022)

The research methodology consists of 1) collecting data on daily rainfall; 2) analyzing the design rainfall using frequency analysis; 3) to analyze distribution of hourly rainfall; 4) to analyze effective rainfall; 5) to analyze hydrograph of design flood as the input to the computational fluid dynamics (CFD).

3. Results and Discussion

3.1. Plan Design

This physical model is based on the results of the plan design with an undistorted scale of 1:50. The test results are due to the flow discharge: $Q_{1.01year}$, Q_{2year}, Q_{5year}, Q_{10year}, Q_{25year}, Q_{50year}, $Q_{100year}$, $Q_{200year}$, $Q_{1000year}$, $Q_{0.5PMF}$, and Q_{PMF}. After evaluating the design flood with the original design condition, it can be concluded as presented in Table 2.

Table 2 Velocity recapitulation in upstream approach channel original design (The authors)

No.	Design flood	V, (m/s)	Conditional, V < 4 m/s	Elevated water level (m)	Freeboard head to the peak dam (m)
1	$Q_{1,01th}$	0.971	Fulfill	326.000	4.000
2	Q_{2th}	0.971	Fulfill	326.550	3.450
3	Q_{5th}	0.971	Fulfill	326.800	3.200
4	Q_{10th}	0.971	Fulfill	326.900	3.100
5	Q_{25th}	1.373	Fulfill	327.050	2.950
6	Q_{50th}	1.373	Fulfill	327.100	2.900
7	Q_{100th}	1.373	Fulfill	327.200	2.800
8	Q_{200th}	1.373	Fulfill	327.275	2.725
9	Q_{1000th}	1.373	Fulfill	327.450	2.550
10	$Q_{0,5PMF}$	1.373	Fulfill	327.600	2.400
11	Q_{PMF}	1.844	Fulfill	329.050	0.950

Based on the data in the table above, it can be concluded that design flood $Q_{1,01year}$ until Q_{PMF}, which is evaluated and has met the requirement of the velocity boundary, and the freeboard is also safe for all discharge conditions [18]. However, the measurement of flow hydraulics in the stilling basin was carried out at the center of 3 measurement points (sections 47-49). Table 3 shows the results of the hydraulic condition measurement at the center of 'damping' pool.

Table 3 Recapitulation of the flow condition in a stilling basin (original design) (The authors)

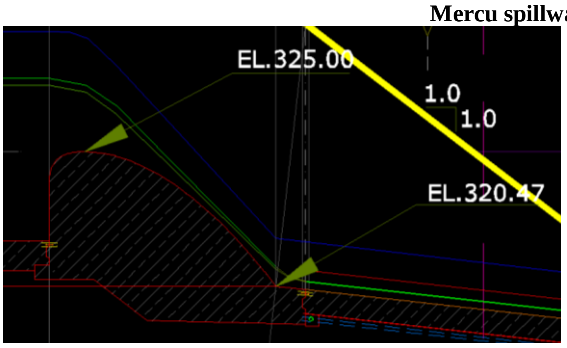
No.	Design flood	Velocity (m/dt)	H water (m)	Froude number	Flow type
1	$Q_{1,01th}$	2.441	2.900	0.454	Sub-critic
2	Q_{2th}	4.188	4.144	0.643	Sub-critic
3	Q_{5th}	5.990	4.450	0.909	Sub-critic
4	Q_{10th}	6.294	4.639	0.936	Sub-critic
5	Q_{25th}	-	5.050	-	-
6	Q_{50th}	-	5.100	-	-
7	Q_{100th}	-	5.156	-	-
8	Q_{200th}	-	5.125	-	-
9	Q_{1000th}	-	5.247	-	-
10	$Q_{0,5PMF}$	-	5.572	-	-
11	Q_{PMF}	-	6.633	-	-

The stilling basin effectively muffled water from the chute way. On the flow discharge $Q_{1,01year}$ until Q_{10year}, there was a sub-critic flow; however, during the flow discharge Q_{25year} until Q_{PMF}, the flow that happened could not be analyzed because it could not be measured due to the air bubble that entered the Pitot tube measurement tool. The observation of the flow hydraulic conditions in the downstream approach channel was carried out at 5 sections, started from downstream 'damping' pool (Sections 50-54).

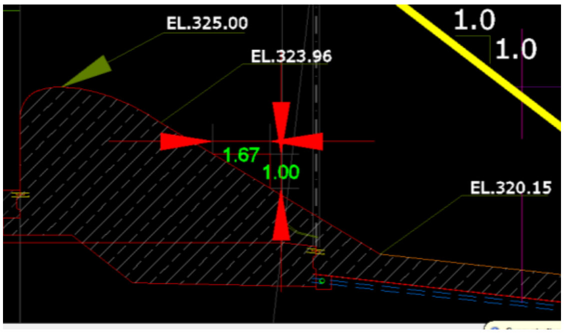
3.2. Design Change

Based on the analysis of the model test with the original design, there is change by changing from the original design at the wall part of the approach channel and downstream slope of the spillway, which is hoped, will produce observations as well as even measurements flow. Fig. 9 presents the wall of approach channel.

Mercu spillway downstream



Condition of the original design at Harold curve dam spillway



The condition of serial 1 at the spillway downstream (Harold curve) changes the slope. At 2.87 m from the spillway crest and downstream, the slope was changed to 1:1.666. The beginning of slope is at an elevation of +323.96, and the ending of slope (beginning chute way I) is at +320.15.

Original design

Serial 1

Fig. 9 Wall of the approach channel (Own study; the authors)

3.3. Model Test of Design Change

Table 4 summarizes the velocity in the upstream approach channel.

Table 4 Velocity recapitulation in upstream approach channel (The authors)

No.	Design flood	V (m/s)	Conditional, V < 4 m/s	Elevated water level (m)	Freeboard to the dam peak (m)
1	$Q_{1,01year}$	0.971	Fulfill	326.025	3.975
2	Q_{2years}	0.971	Fulfill	326.550	3.450
3	Q_{5years}	0.971	Fulfill	326.808	3.192
4	$Q_{10years}$	0.971	Fulfill	326.833	3.167
5	$Q_{25years}$	0.971	Fulfill	327.025	2.975
6	$Q_{50years}$	0.971	Fulfill	327.100	2.900
7	$Q_{100years}$	0.971	Fulfill	327.200	2.800
8	$Q_{200years}$	1.373	Fulfill	327.250	2.750
9	$Q_{1000years}$	1.239	Fulfill	327.392	2.608
10	$Q_{0,5PMF}$	1.681	Fulfill	327.517	2.483
11	Q_{PMF}	2.801	Fulfill	328.917	1.083

Based on the table above, it can be concluded that the design flood $Q_{1,01year}$ until Q_{PMF} that is evaluated has met the velocity boundary requirement, and the freeboard is also safe under all discharge conditions. There is less difference in the water level head change caused by the change in the left and right sides of the approach channel wall. Table 5 presents a recapitulation of the flow conditions in the upstream channel Serial-1.

Table 5 Recapitulation of flow conditions in upstream channel Serial-1 (The authors)

No.	Design flood	Velocity (m/s)	H air (m)	Froude number	The type of flow
1	$Q_{1,01year}$	1.131	2.825	0.223	Sub-critic
2	Q_{2years}	1.226	3.807	0.207	Sub-critic
3	Q_{5years}	1.294	4.092	0.209	Sub-critic
4	$Q_{10years}$	1.252	4.308	0.197	Sub-critic
5	$Q_{25years}$	1.474	4.705	0.223	Sub-critic
6	$Q_{50years}$	1.520	4.845	0.225	Sub-critic
7	$Q_{100years}$	1.370	4.922	0.200	Sub-critic
8	$Q_{200years}$	1.337	4.948	0.195	Sub-critic
9	$Q_{1000years}$	1.697	5.352	0.236	Sub-critic
10	$Q_{0,5PMF}$	1.740	5.471	0.240	Sub-critic
11	Q_{PMF}	2.896	8.473	0.318	Sub-critic

Based on the above table, the type of flow at test discharge $Q_{1,01year}$ until Q_{PMF} is sub-critic. The stilling basin from ‘damping’ pool and the meeting with the actual river water level head (TWL) had good effects on the flow conditions in the escape channel.

4. Conclusion

Based on the analysis result and model test, we can conclude that the original design and model test showed that the water head over the checkbox measure tool during calibration only decreased by 1 to 3 mm from the water level head of the checkbox analysis. In addition, the relative error at discharge calibration of the water level head over the spillway was not more than 2%, and the highest error was 1.07% for a 10-year discharge.

During the discharge calibration, there was a flow with a slight cross-over after the spillway because the wall in the left part of the approach channel was of less length. However, the negative pressure (cavitation) is still under safe requirements: 4 m, and the highest negative pressure is about -1.75 m during PMF discharge.

The damping at all flow discharges happened in the damping space, so there was no problem at the end channel. The stilling basin effectively damps water from Chute Way III.

References

[1] BACHTIAR S., LIMANTARA L.M., SHOLICHIN M., and SOETOPO W. Optimization of integrated reservoir for supporting the raw water supply. *Civil Engineering Journal*, 2023, 9(4): 860-872.

[2] ASMELITA, LIMANTARA L.M., BISRI M., SOETOPO W., and FARNI I. Rice self-sufficiency and optimization of irrigation by using system dynamic. *Civil Engineering Journal*, 2024, 10(2): 489-501.

[3] SATYAGRAHA B., BISRI M., LIMANTARA L.M., and ANDAWAYANTI U. Model of water value optimization based on land use change. *Journal of Water and Land Development*, 2017, 36(I-III): 143-152.

[4] MULYONO J. Konsepsi keamanan bendungan dalam pembangunan dan pengelolaan bendungan. *Jurnal Infrastruktur*, 2017, 3(1): 1–62.

[5] TAUFI M. Pelimpah samping studi kasus pada pelimpah bendungan bayang-bayang Kabupaten Bulukumba. *Jurnal Teknik Pengairan*, 2012, 2(1).

[6] ANDAR J.H., and PAULUS N. Tinjauan jarak awal loncat air akibat perletakan end sill pada pintu air geser tegak (Sluice Gate). *Majalah Ilmiah UKRIM*, 2007, 2: 1-22.

[7] EL-SAYED M., and EL-MAHDY. Experimental method to predict scour characteristics downstream of stepped spillway equipped with V-Notch end sill. *Alexandria Engineering Journal*, 2021, 60(5): 4337-4346. https://doi.org/10.1016/j. aej.2021.03.018

[8] KEIHANPOUR M., and SAMANI A.K. Effects of modern marguerite-shaped inlets on hydraulic characteristics of swirling flow in shaft spillways. *Water Science and Engineering*, 2021, 14(3): 246-256. https://doi.org/10.1016/j. wse.2021.08.005

[9] ALMELAND S.K., MUKHA T., and BENSOW R.E. An improved air entrainment model for stepped spillways. *Applied Mathematical Modelling*, 2021, 100: 170-191. https://doi.org/10.1016/j. apm.2021.07.016

[10] RISDIYANTO A., and PRAWITO A. Dimension analysis of the emergency spillway of Tirawan DAM with the application of the system dynamic model. *Ukarst,* 2022, 6(1). http://dx.doi.org/10.307 37/ukarst. v6i1

[11] PRASETYORINI L., and PRIYANTORO D. Penggunaan stilling basin tipe Bremen modifikasi pada pelimpah Bendungan Tugu di Kabupaten Trenggalek. *Jurnal Teknik Pengairan*, 2015, 6(1): 116-124.

[12] GHADERI A, ABBASI S., ABRAHAM J., and AZAMATHULLA H.M. Efficiency of trapezoidal labyrinth shaped stepped spillways. *Flow Measurement and Instrumentation*, 2020, 72: 101711. https://doi.org/10.1016/j.flowmeasinst.2020.101 711

[13] ASL J.J., SEGHIER M.E.A.B., OHADI S., and VAN GELDER P. Efficient method using whale optimization algorithm for reliability-based design optimization of labyrinth spillway. *Applied Soft Computing Journal*, 2021, 101: 107036. https://doi.org/10.1016/j. asoc.2020.107036

[14] DAMARNEGARA A., WARDOYO W., PERKINS R., and VINCENT E. Computational fluid dynamics (CFD) simulation on the hydraulics of a spillway. *IOP Conference Series: Earth and Environmental Science*, 2019. DOI: 10.1088/1755- 1315/437/1/012007

[15] HUANG J., LU J., LI R., FENG J., and YANG H. Numerical study on the cumulative effect of supersaturated TDG through the spillway. *Ecohydrology & Hydrogeology*, 2021, 21(2): 292-298. https://doi.org/10.1016/j. ecohyd.2021.01.003

[16] MOOSELU M.G., NIKOO M.R., BAKHTIARI P.H., RAYANI N.B., and IZADY A. Conflict resolution in the multi-stakeholder stepped spillway design under uncertainty by Machine Learning Techniques. *Applied Soft Computing*, 2021, 110: 107721. https://doi.org/10.1016/j. asoc.2021.107721

[17] KARIM R.A., and MOHAMMED J.R. A comparison study between CFD analysis and PIV technique for velocity distribution over the standard ogee crested spillways. *Heliyon*, 2020, 6(10): e0515. https://doi.org/10.1016/j. heliyon. 2020.e05165

[18] HU H.H. Computational fluid dynamics. In: *Computer Science Handbook*. 2nd ed. Elsevier, 2012. https://doi.org/10.1017/cbo9780511780066

[19] MENTERI PEKERJAAN UMUM DAN PERUMAHAN RAKYAT REPUBLIK INDONESIA. *Peraturan Menteri Pekerjaan Umum dan Perumahan Rakyat Nomor 04/PRT/M/2015 Tahun 2015 tentang Kriteria dan Penetapan Wilayah Sungai*. Kementerian Pekerjaan Umum dan Perumahan Rakyat, Jakarta, 2015. https://peraturan.bpk.go.id/Details/159834/permen-pupr-no-04prtm2015-tahun-2015

参考文:

[1] BACHTIAR S.、LIMANTARA L.M.、SHOLICHIN M. 和 SOETOPO W. 支援原水供應的綜合水庫最佳化。土木工程學報, 2023, 9(4): 860-872。

[2] ASMELITA、LIMANTARA L.M.、BISRI M.、SOETOPO W. 和 FARNI I。土木工程學報, 2024, 10(2): 489-501。
[3] SATYAGRAHA B.、BISRI M.、LIMANTARA L.M. 和ANDAWAYANTI U.基於土地利用變化的水價值優化模型。水土資源開發, 2017, 36(I-III): 143-152.
[4] MULYONO J. 大壩建設與管理中的大壩安全理念。基礎設施雜誌，2017，3(1)：1－62。
[5] TAUFI M. 側面溢洪道布魯庫巴攝政影子大壩溢洪道案例研究。《技術雜誌》，2012，2(1)。
[6] ANDAR J.H. 和 PAULUS N. 垂直滑動水門上端坎放置引起水躍初始距離的測量（閘門）。英国后勤组织科學雜誌，2007年，2：1-22。
[7] EL-SAYED M. 和 EL-MAHDY。預測V型缺口端坎階梯式溢洪道下游冲刷特性的實驗方法亞歷山大工程雜誌，2021，60(5)：4337-4346。https://doi.org/10.1016/j.aej.2021.03.018
[8] KEIHANPOUR M. 和 SAMANI A.K.現代瑪格麗特形進水口對豎井溢洪道旋流水力特性的影響水科學與工程, 2021, 14(3): 246-256. https://doi.org/10.1016/j.wse.2021.08.005
[9] ALMELAND S.K.、MUKHA T. 和 BENSOW R.E. 階梯式溢洪道的改良空氣夾帶模型。應用數學建模，2021，100：170-191。https://doi.org/10.1016/j.apm.2021.07.016
[10] RISDIYANTO A. 和 PRAWITO A. 應用系統動力學模型對蒂拉萬大壩緊急溢洪道進行尺寸分析。喀斯特，2022，6（1）。http://dx.doi.org/10.307 37/ukarst. v6i1
[11] PRASETYORINI L. 和 PRIYANTORO D. 在特倫加勒克縣紀念碑大壩溢洪道上使用改良的不來梅型消力池。水利學報, 2015, 6(1): 116-124。
[12] GHADERI A、ABBASI S.、ABRAHAM J. 與 AZAMATHULLA H.M.梯形迷宮形階梯溢洪道的效率。流量測量與儀表，2020，72：101711。
[13] ASL J.J.、SEGHIER M.E.A.B.、OHADI S. 和 VAN GELDER P. 使用鯨魚優化演算法進行迷宮式溢洪道基於可靠性的設計優化的有效方法。應用軟體運算雜誌，2021 年，101：107036。https://doi.org/10.1016/j.asoc.2021.107721
[14] DAMARNEGARA A.、WARDOYO W.、PERKINS R. 和 VINCENT E. 溢洪道水力學的計算流體力學模擬。物理研究所會議系列：地球與環境科學，2019。
[15] 黃靜，魯靜，李瑞，馮靜，楊浩。生態水文與水文地質, 2021, 21(2): 292-298. https://doi.org/10.1016/j.ecohyd.2021.01.003
[16] MOOSELU M.G.、NIKOO M.R.、BAKHTIARI P.H.、RAYANI N.B. 和 IZADY A。應用軟體運算，2021，110：107721。https://doi.org/10.1016/j. asoc.2021.107721
[17] KARIM R.A. 和 MOHAMMED J.R. 差价合约分析與粒子成像技術之間標準反角溢洪道速度分佈的比較研究。太阳神，2020，6(10)：e0515。https://doi.org/10.1016/j.heliyon. 2020.e05165
[18] HU H.H. 計算流體動力學。請參閱：計算機科學手冊。第二版。愛思唯爾，2012。
[19] 印度尼西亚人民共和国议员。公共工程和公共住房部2015年第04/前列腺癌逆转录酶/米/2015号法规，涉及河流区域的标准和确定。公共工程和人类住区部长，雅加达，2015 年。https://peraturan.bpk.go.id/Details/159834/permen-pupr-no-04prtm2015-tahun-2015