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EXPERIMENTAL STUDY ON THE SIDE CHANNEL SPILLWAY AND ITS IMPACT ON THE JUMP, CROSS FLOW AND ENERGY DISSIPATION

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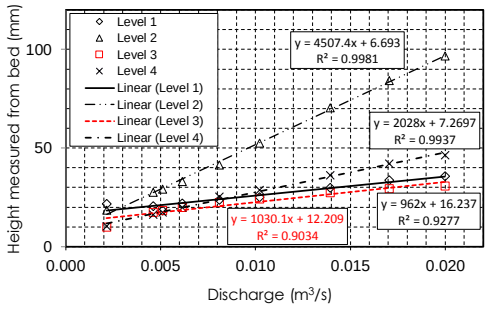
Graphical abstract



Abstract

The utilization of the side channel spillway as the primary component of dam is generally due to the limitation of the available space to construct conventional spillway. Some impacts may only be identified through the hydraulic physical model study; these include the presence of the chaotic jumps, the cross flow, as well as the performance of the energy dissipation. This paper presents the result of the experimental study of three-dimensional behaviour of flow over the entire components of the side channel spillway of Bener Dam, Indonesia. The main dam and its appurtenant components were built, and various discharges were introduced to study the hydraulic performance of the spillway crest, the stilling basin, the chute, and the energy dissipater. The results show that firstly, some chaotic hydraulic jumps were found at the stilling basin at downstream spillway crest. These chaotic hydraulic jumps would produce significant vibration that may endanger the nearby structures. Secondly, the presence of the cross flow along the steep channel downstream of the stilling basin may also need to be eliminated to reduce its impact on the rise of water surface level. Thirdly, the presence of the hydraulic jumps at the energy dissipater basin has proven that the energy dissipater has performed well where local scour around the downstream structure was found to be not significant. To anticipate the raising of the water surface elevation at the energy dissipater basin, increasing the elevation of energy dissipater wall from +212.50 m to +215.00 m is highly recommended.

Keywords: Side channel spillway, chaotic jumps, cross flow, energy dissipation

1.0 INTRODUCTION

1.1 General

The Bener Dam is one of several dams that is built in the province of Central Java, Indonesia. The purpose of the dam is to fulfill the supply water for the development of the new international airport of Yogyakarta. Bener Dam is laid in Central Java (see Figure 1), at the position of 7°36'0"S and 110°1'14"E. The detailed design of Bener Dam was prepared in 2011, and the construction will be located at the upstream of Bogowonto River. The River originates from the Kedu area, flows to the south and ends in the Indonesian Ocean. The Bogowonto watershed is one of some watersheds in the Serayu-Bogowonto River Basin Management, with a catchment area of 605.91 km^2 and potential water discharge of 9.694 m^3/s. Eastern area of Bogowonto Watershed is bordered with Serang Watershed, as for the western part of the Bogowonto Watershed is bordered with Cokroyasan Watershed.

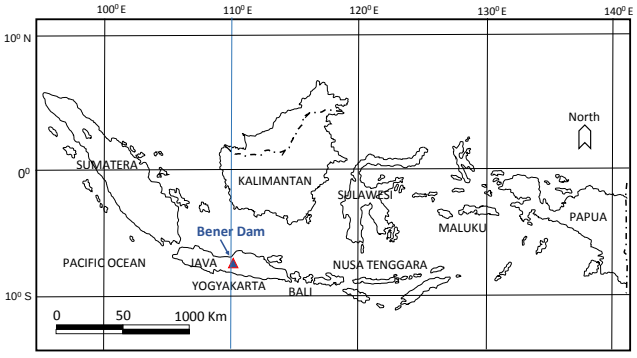


Figure 1 Location of Bener Dam

A study of physical hydraulic models needs to be carried out to evaluate the design of the Bener Dam. Physical model studies on the behavior of flow over the side-channel spillway have indicated the significant difference between the behavior obtained from the real practice and the physical model studies [1, 2, 3, 4]. This evaluation is intended to discover the reliability of the auxiliary structure and its hydraulic performances, such as spillway, energy dissipator, flow pattern on the downstream area of the spillway, and the scour pattern on the downstream area of the energy dissipator. Those evaluations were conducted on various discharge type, which related to the return period or Probable Maximum Flood (PMF).

The ability of a dam to reduce the peak elevation of a flood would highly depend on the storage characteristic of the constructed dam [5, 6]. The geometry of the spillway (crest width and crest elevation) would also affect the dissipation energy that would be achieved. Considering that the dam elevation is relatively high (on +356.00 m), it is potentially that the flood dissipation would have minimal value. Figure 2 shows the storage characteristic curve of Bener Dam, in which the volume is about 100 million m^3 at an elevation of +365.00 m.

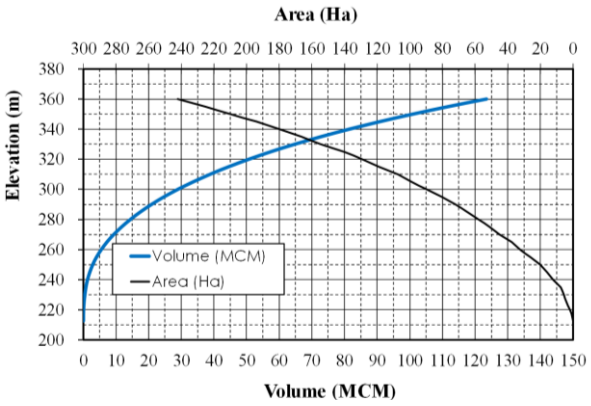


Figure 2 Storage Characteristic of Bener Dam

1.2 Dam Spillway Capacity

It is required that under the storage capacity of Dam Bener, as shown in Figure 2, the spillway should convey the discharge at a particular design flood. The dam spillway capacity should have sufficient capacity to avoid the overtopping through the dam crest, by releasing some amounts of excess water (flood) from the inside into the outside of the reservoir [7]. Excess water from the reservoir is released out through the spillway, to be returned to the river on the downstream. The safety factor is an essential value for a spillway design, given that dam break could occur because of inaccurate hydraulic design of the spillway capacity to release excess water. Sufficient overflow capacity is a primary requirement in spillway design for embankment dam (either the earth-fill dam or rock-filled dam) because the overtopping through dam body might cause collapse to the dam. Meanwhile, the concrete dam is relatively more resistant to a shallow overtopping through the dam body. Additional cost for dam construction is usually not linear with the increase of the spillway capacity. Also, it needs to be understood that building a dam spillway with an adequate capacity will not increase the cost (relatively) to the spillway capacity of the large dam. Either hydraulically or structurally, the spillway must be safe and must be appropriately placed so that the flow would not cause erosion to the downstream area [8, 9].

Various hydraulic simulation techniques were developed in order to observe the hydraulic behaviour on the form of flow pattern of the spillway, whether on the spillway crest, channel or the energy dissipater [10, 11]. According to the safety criteria for the overflow hazard caused by flood, such hydraulic behaviour must be able to describe accurate

information about flow profiles. Due to the geometry of the spillway structure is generally three-dimensional, the understanding of theoretical analysis faced the uncertainty. Because the theoretical approach is less promising, then the study of physical models needs to be carried out. Another structural requirement is that the spillway surface must be sufficiently resistant to potential erosion due to the relatively high scour velocity. The high scour velocity can be caused by the water jump (from the reservoir water level to the downstream water level). Technical data of the Bener Dam plan and related data to spillway structure design are shown in Table 1. The design flood that is used to check the dam spillway capacity is the Probable Maximum Flood Discharge (Q_{PMF}). The Q_{PMF} is basically a flood with a small probability of exceedance (at the degree of less than 0.01%) or return period of higher than 10,000 years.

Table 1 Prototype and Model Dimension

No.	Parameter	Unit	Prototype	Model
1	Length of spillway channel	m	300	3.00
2	Width of river at near downstream of energy dissipator	m	60	0.60
3	Width of side channel spillway	m	80	0.80
4	Width of spillway channel	m	20	0.20
5	Flood discharge with 1000 years return period	m³/s	1018	0.0102
	a. Height of water above spillway crest	m	3.26	0.03
	b. Flow velocity at spillway channel	m/s	3.91	0.39
6	Flood discharge with Probable Maximum Flood (PMF)	m³/s	1996	0.020
	a. Height of water above spillway crest	m	5.14	0.05
	b. Flow velocity at spillway channel	m/s	4.86	0.49

1.3 Reservoir Routing

The storage accumulation on a reservoir depends on the difference between the inflow and outflow velocity [12, 13]. For discrete time Δt, the above statement could be written in mathematical form as Equation (1).

$$\Delta S = Q_i \Delta t - Q_o \Delta t \quad (1)$$

where ΔS is water storage change, Q_i is average inflow at time Δt (m³/s), and Q_0 is average outflow at time Δt (m³/s).

Considering the highly dynamic flow, the occurred stress was not evenly distributed on the main direction of flow, either against the function of time or space. The solution of Equation (2) is then carried out using input data in the form of reservoir characteristic, flow hydrograph on certain return period, and the spillway geometry. Hydrograph of peak flow on various return periods on Bener Dam is shown in Figure 3. These inflow hydrographs are taken from the previous study that was carried out by PT. Indra Karya of Engineering Consultant [14].

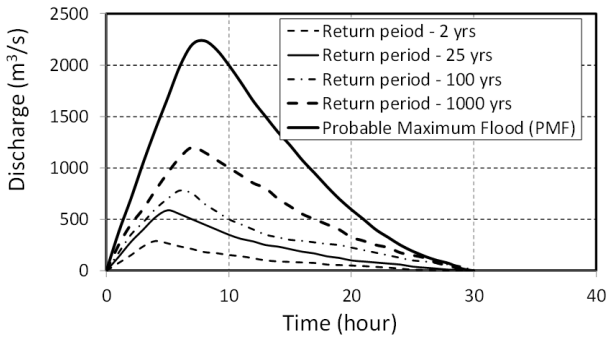


Figure 3 Flood hydrographs at various return periods

The Bener Dam was designed to use side channel spillway, with a total width of 80.00 m, and crest elevation at + 350.00 m. Such spillway layout was intended to obtain a maximum dissipation of peak flow with minimum space requirements. The dissipation efficiency of the probable maximum flood (PMF) with such side channel spillway geometry was of 2.70%.

2.0 METHODOLOGY

2.1 Scale of the Hydraulic Model Test

The model should be as large as possible to resemble the prototype and gain the high accuracy of the simulation results. On the other side, the determination of model scale should be based on the laboratory condition, both in term of space and facilities. Also, the determination of model scale has to consider the similarity of the Froude number between the model and the prototype. By determining the model scale for length n_L, the scale for other hydraulic parameters is presented in Table 2.

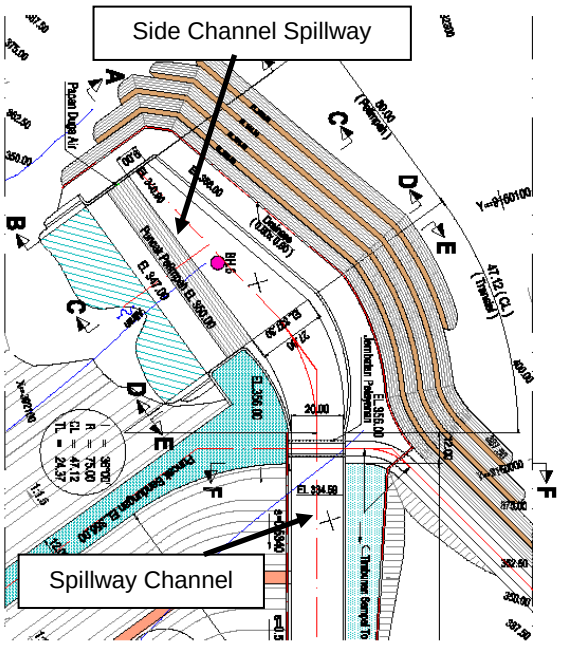
Table 2 Hydraulic parameters scale length n_L

Parameters	Notation	Model Scale
Length	n_L	n_L
Depth	n_L	n_L
Area	n_A	n_L^2
Volume	n_V	n_L^3
Time	n_T	$n_L^{0.5}$
Velocity	n_U	$n_L^{0.5}$
Discharge	n_Q	$n_L^{2.5}$

Through consideration of the laboratory condition, both space and supporting facilities, the reasonable scale of the model was 1:100 or n_L = 100. The hydraulic parameters of Bener Dam model for 1:100 scale is shown in Table 1.

The dam model that built in the Hydraulics Laboratory of the Civil and Environmental Engineering Department FT-UGM with a non-distortion scale of 1: 100 occupied a space of 30.00 m x 12.00 m.

The side channel spillway in the model was constructed with an acrylic material and consists of Ogee-typed crest, chute channel, stilling basin, and the energy dissipator. The following Figure 4 and 5 show the construction stages of the side channel spillway and spillway channel.



(a) Schematic (plan view) of the Spillway Design

(b) Physical Model Preparation

Figure 4 Preparation of side channel spillway model

Figure 5 Preparation of the spillway channel model

2.2 Instrumentation of the Hydraulic Model Test

2.2.1 Discharge Measurement

The measurement for the discharge in the model test was conducted using V-notch sharp-crested weir with θ = 90°, which commonly called Thomson weir. Calibration for Thomson weir needed to be undertaken in order to find the overflow coefficient, which could be varied due to the imperfection of the weir manufacture or installation process. Figure 6 and 7 shows the sketch and the analysis result of Thomson weir calibration.

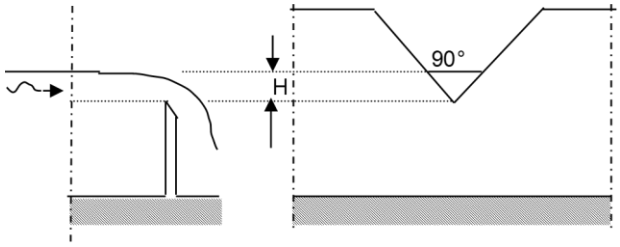


Figure 6 Sketch of flow over Thomson weir

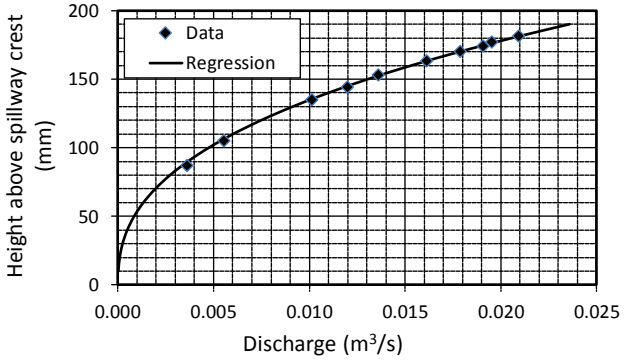


Figure 7 Calibration of Thomson weir

The flow discharge over the Thomson weir is generally expressed as the following equation:

$$Q = Cd \frac{8}{15} \tan \frac{\theta}{2} \sqrt{2g} H^{5/2} \quad (2)$$

where Q is discharge over the weir crest (m^3/s); θ is the angle of the weir, g is acceleration due to gravity (9.78 m/s^2), and H is the height of overflow (m).

2.2.2 Water Depth Observation

The measurement of the flow depth was carried out by using a specially designed electronic device, namely 'Level'. Four Levels' were assembled to record the water depth at different locations in the model. The 'Level' functions as measuring the frequency of the conductor capacity of a submerged cable. The frequency value would depend on the water depth of which the cable submerged. The accuracy tests on the performance of the 'Level' were applied in still water conditions, and fluctuations reading were tested for acceptability of the information produced. The rate of sampling of the flow meter is 1 second. The accuracy test of the 'Level' (Level 1, Level 2, and Level 3) through the 89 recorded data of the frequency reading showed that more than 96% output data were on accuracy range of +2%. Table 3 shows the information that fulfilled the specified accuracy range. The 95% as a minimum required degree of accuracy is considered acceptable for the hydraulics instrumentation [4].

Table 3 Accuracy of water surface level sensor

Sensor	Degree of Accuracy				
	± 0.1%	± 0.5%	± 1%	± 2%	± 5%
Level 1	100%	100%	100%	100%	100%
Level 2	99%	100%	100%	100%	100%
Level 3	4%	36%	52%	96%	100%

If the entire 89 data were processed according to normal distribution equation as in Equation (3), the result showed the reliability of the sensor (see Figure 8). The normal distribution equation is written as follows.

$$Y = \frac{e^{-\frac{1}{2}x^2}}{\sigma\sqrt{2\pi}} \quad (3)$$

where X is the value of $Data - Data_{Average}$; and σ is the standard deviation of data value.

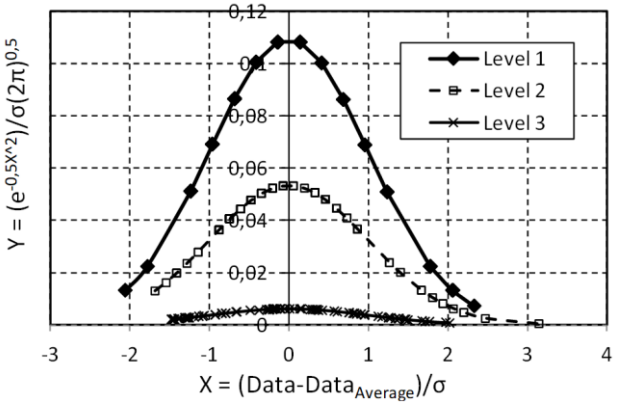


Figure 8 Distribution showing the data reliability

2.3 Calibration of Sensors

The experimental study on hydraulic performance of the dam and its auxiliary components was conducted with four units of 'Level'. Previously, the 'Level' needs to be calibrated to discover its correlation with actual water depth. The result of 'Level' calibration is shown in Figure 9.

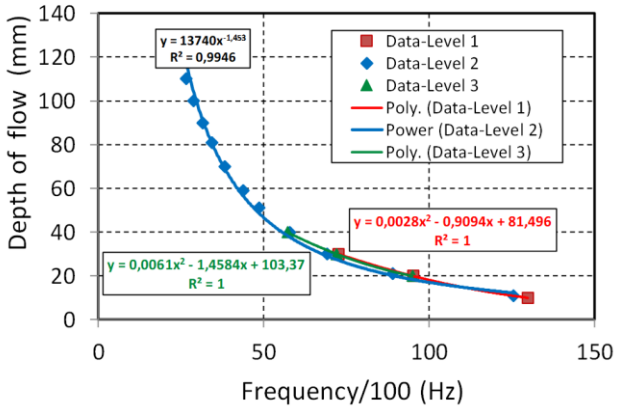


Figure 9 Calibration of water depth sensors

2.4 Scenario of Hydraulic Model

As previously explained, the testing was only varied on discharge, while the model constructions are fixed. The variation of discharges consists of 9 types. With the model being set at a non-distortion scale of 1:100, the conversion of the discharge rate on the prototype was by multiplying with discharge scale factor, which is $n_Q = 100^{(5/2)} = 100{,}000$. The nine discharge variations with discharge rate value on the model and on the prototype are shown in Figure 10.

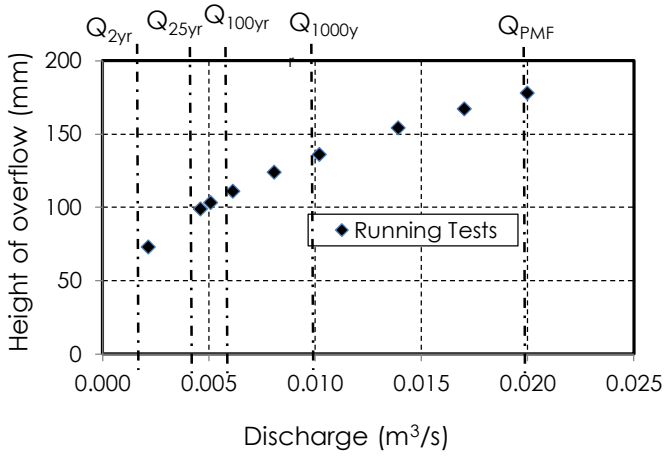


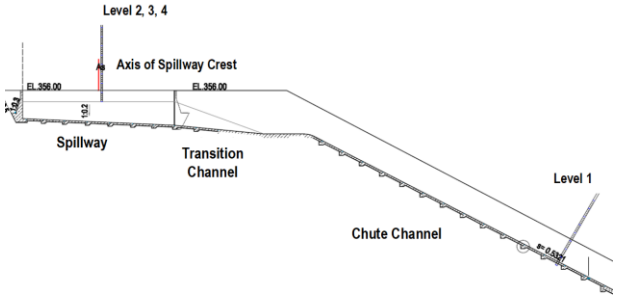
Figure 10 Various discharges during the test

2.5 Running of Hydraulic Model Test

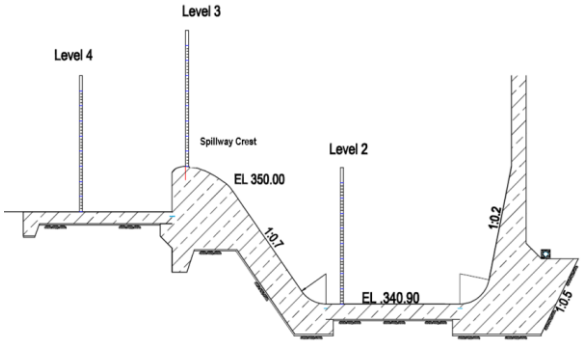
2.5.1 Water Depth at Four Locations

The position of the water depth observation is shown in Figure 11. The monitoring of the flow depth on four locations for all nine discharges showed the consistency of the correlation between discharge and flow depth. The graph of the recorded data of the monitoring is shown in Figure 12.

Minimum flow depth on the channel location (Level 1) was +20 mm, and occurred on the discharge of 4.20 l/s. At the area of the stilling basin (Level 2), there was a significant increase (ranged from 1.95 – 2.00 times the flow depth on the reservoir) for the various increase of the discharges. It happened due to the stilling basin area on the downstream sector of the spillway crest was inadequate to accommodate the water volume spilled out from the side channel spillway. Relatively, there is only little increase of flow depth at locations above the crest and in reservoir pond (Level 3 and Level 4 respectively) for various increases in discharges. The flow depth measured from the spillway crest on Level 3 generally ranged from 0.55 – 0.85 (or in average = 0.78) times the flow depth on the reservoir (Level 4).



a) The longitudinal section over the spillway structure



b) Cross section over spillway structure

Figure 11 Position of water depth observation

The flow depth over the side channel spillway is strongly influenced by spillway geometry, i.e., crest shape, surface roughness, and crest width [15, 16, 17]. The spillway geometry includes the shape and length of the spillway crest and the elevation of the channel bed at the upstream side of the spillway.

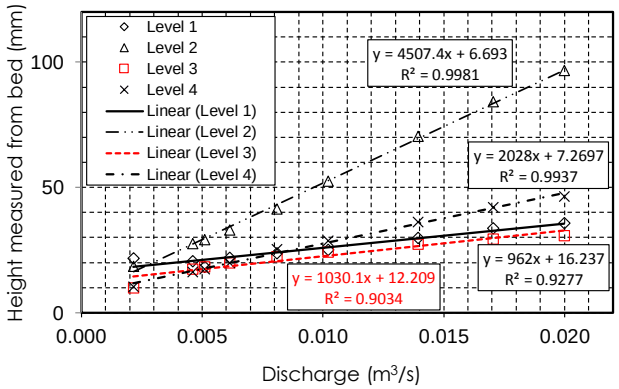


Figure 12 Discharge vs. depth of water at four different locations

The geometry may significantly affect the capacity of the spillway to releasing the discharge from the reservoir through the spillway. The water depths obtained from the model and its conversion to the water surface elevation at the prototype is shown in Figure 13.

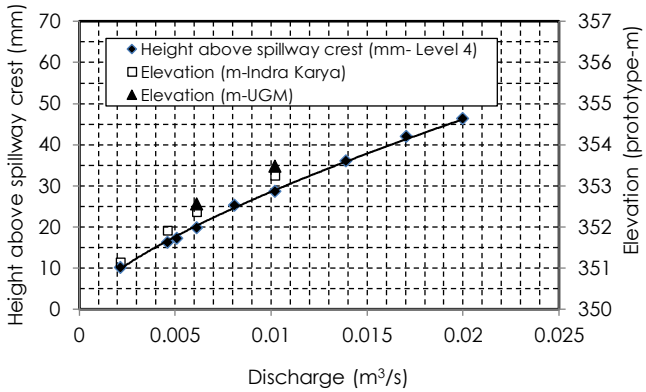


Figure 13 Water surface elevation obtained from the model and analysis

It is seen from Figure 13 that the water surface elevation, as observed in the model, is having close value with that obtained from the previous study [14]. The maximum discrepancy between those two was found at approximately 0.14 %.

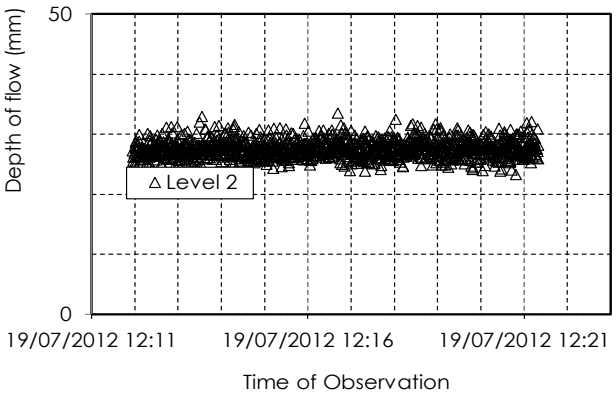
2.5.2 Jumps at Downstream of the Spillway Crest

The spillway can flow Q9 flood discharge of 19.96 l/s on the model or 0,020 m^3/s on the prototype, which was equal with Q_{PMF} = 1,996 m^3/s, with adequate freeboard. The water level at the upstream area of spillway reaches elevation +352.88 m for a 1000-years return period discharge (Q_{1000th}), while for Q_{PMF} it reaches elevation +354.64 m. Based on the elevation

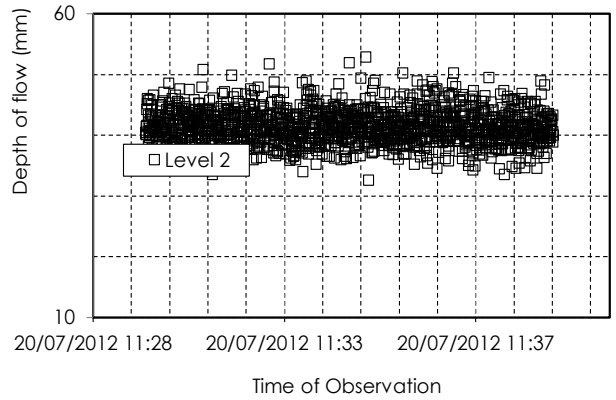
of the dam top +356 m, then the heights of the freeboard from the spillway crest are 3.12 m and 1.36 m for Q_{1000th} and Q_{PMF} respectively. Those meet the standard criteria of the freeboard for each return period discharge.

The results of flow depth measurement in the stilling basin area (Level 2) show the irregularity of the water depths with high turbulence in this area. Such condition may require serious attention to cope with possible vibrations that may endanger the structures.

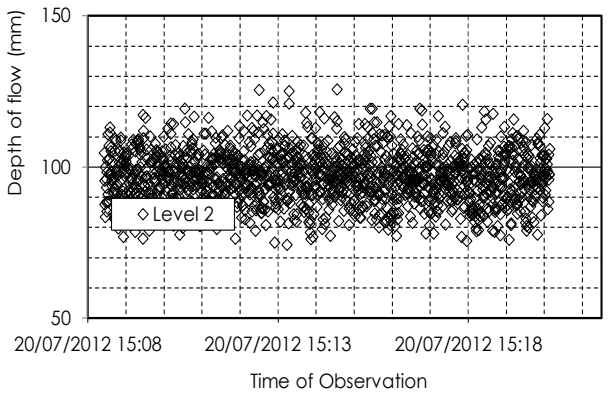
Results from the Level 2 measurements for discharge Q2, Q5, and Q9 can be seen in Figure 14 a), b), and c) respectively.



(a) Chaotic jumps at Q2 scenario discharge



(b) Chaotic jumps at Q5 scenario discharge



(c) Chaotic jumps at Q9 scenario discharge

Figure 14 Chaotic jumps at downstream of the side channel spillway

2.5.3 Transition Channel

At the downstream area of the side channel spillway, before entering the chute channel, a section called transition channel is commonly introduced [18, 19, 20]. From the transition channel, the flow from the upstream area of the spillway was evenly distributed to the spillway. After the water overflows from the spillway crest, the flow chutes as supercritical flow, then hydraulic jumps occurred on spillway trough and transition channel [21, 22]. At low discharge, water jump exists in the transition channel before entering the chute channel. The existence of water jump at low discharge was due to the downstream area of the transition channel has zero slopes, which then resulted in generating the subcritical flow before entering the chute channel 9 [23]. At the medium discharge of Q = 0.006 m^3/s, the water jump occurred at the base of spillway's downstream. This is due to there was a vertical wall at a distance between 9 – 27 m on the opposite of the spillway. This wall connected with the transition channel wall, which has a curved shape. This curve wall deflected the flow direction toward the spillway channel. Furthermore, at the higher discharge of Q = 0.010 m^3/s and 0.020 m^3/s, the hydraulic jump that occurred at near downstream of spillway channel was significantly high and reached almost the top of the wall of the spillway channel. Therefore, for Q_{PMF}, the fluctuation of the hydraulic jump could reach crest of the wall elevation, particularly at the upstream area. This occurred because of the 9.00 m length of the transition channel is apparently too short.

2.5.4 Spillway Channel

The spillway channel is basically a part of the spillway that has the function as conveying the discharge from the reservoir to the river reach at the near downstream of the dam. Since normally the channel is steep, the flow velocity in this area could be very high. The irregularity of the flow pattern along this steep channel would create a cross flow. During the entire runs, such cross flow appeared on the spillway channel, particularly at low discharges, as shown in Figure 15. The objection regarding the presence of the cross flow persists in the form of the possible rise in the water surface elevation along the spillway channel.

Figure 15 Cross flow along the steep channel

2.5.5 Energy Dissipator

The energy dissipator used is a combination of a stilling basin and apron at the downstream. With this type of energy dissipator, it is expected that there will be a hydraulic jump in the stilling basin at a small discharge, and water jump from chute channel to downstream at high discharge.

The following is a description of the flow conditions in the energy dissipator for various discharge values.

[1] At low discharge Q_{2yr}, water jump has occurred in the stilling basin. Therefore, the stilling basin functioned perfectly as energy dissipator. For larger discharge Q_{100yr}, the stilling basin still worked to form a water jump, yet the water spilled through the stilling basin's wall. Thus, the height of the stilling basin wall needs to be increased.

[2] At large discharges, Q_{1000yr}, the water has jumped to the downstream of the stilling basin, where the water jump is in the energy dissipator channel located downstream of the stilling basin. The left side of the Figure shows the photo from the side part that displays the flow of the water jump to the energy dissipator channel. The right photo shows the flow condition as seen from the downstream. Concrete blocks of size 2 x 2 x 1 cm^3 were installed at the downstream area of the energy dissipator channel. Some anticipation is needed to reduce the enormity of local scouring that occurs at the downstream of the energy dissipator structure.

[3] At Q_{PMF}, the water has jumped further downstream area of the energy dissipator. The flow condition at this discharge type is explained as follows. The left picture is the documentation photo taken from the side part, which shows the water flow jumped further to the downstream area of energy dissipator, that reached distance about 80 cm from the downstream area of the energy dissipator (equal 80 m on the prototype). Some of the concrete blocks have been transported to downstream, indicating that the flow occurred in the downstream area of the energy dissipator potentially cause local scouring. However, the location of the local scouring is distant toward downstream of the energy dissipator, thus did not endanger the stability of the structure. Meanwhile, the water jump occurred near the service bridge and at the downstream of the exhaust duct of the power plant. Therefore, further study is needed to investigate the effect of local scouring that occurred at this location toward the stability of the two structures.

3.0 RESULTS AND DISCUSSION

3.1 Spillway Capacity

The spillway could flow Q_{PMF} and Q_{1000yr} with adequate freeboard. The water surface elevation at the upstream of spillway reached an elevation of +352.88 m for Q_{1000yr}, and elevation of +354.64 m for Q_{PMF}. With a peak elevation of the dam at +356 m, the freeboard was 3.12 m for Q_{1000yr}, and 1.36 m for Q_{PMF}. Its values are still considered safe for structures stability.

The result of flow routing through the spillway channel showed that flow depth ranged from 1.00 – 4.60 m with a velocity of 8.8 m/s at the beginning of chute channel to 38 m/s at the end of spillway channel. The flow velocity range was between 3.15 m/s up to 17.30 m/s. The flow depth observation at the middle part of the chute channel showed a depth of 35.65 mm (equal with 3.565 m on the prototype) with a velocity of 2.8 m/s (equal with 28 m/s on the prototype). The high velocity of the flow would cause considerable shear stress on the surface of the channel bed. Therefore, it needs more attention to the possible threat of the concrete surface peeling.

3.2 Spillway Channel Flow Behaviour

Hydraulic jumps occurred at the location between the near downstream of the side channel spillway and the upstream of spillway channel. At high discharge, the hydraulic jumps occurred right at near the downstream of the spillway, which could reach the crest elevation. This condition could raise water surface elevation above the crest. This happened because the wall location was too close with the spillway (i.e., 9 m).

Visually, it appeared the cross flow occurred on the chute channel, especially at low discharges. This cross flow was caused by the uneven flow that passed the transition channel. Uneven flow occurred due to the flow bent by the transition channel wall and the formation of water jumps and subcritical flow in the transition channel.

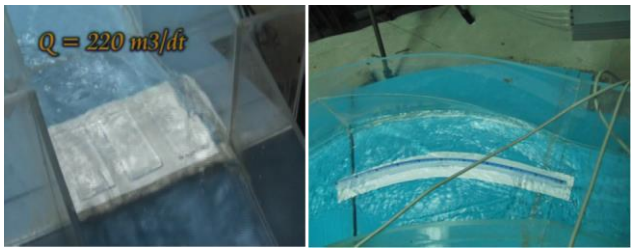
3.3 Performance of Energy Dissipation

Energy dissipator could function optimally by forming water jump on the stilling basing for small discharges up to Q_{100yr}. For Q_{1000yr}, the water jumped to the downstream area of the stilling basin. The water jump existed on the energy dissipator channel located on the downstream area of the stilling basin. As for Q_{PMF}, the water jumped to the downstream area of the energy dissipator. It also indicated the potential of local scouring on the downstream area of the energy dissipator, particularly at high discharge. At the discharge of Q5 (equal with Q_{100yr} and Q_{1000yr}) and Q6 (equal with Q_{1000yr}), it was seen that the flow depth on the stilling basin was very high. Several flows

showed considerable turbulence which then generated overflow above the stilling basin's wall. Vice versa, no overflow occurs through the basin's wall beyond the Q5 and Q6. Erodibility of the riverbed on the downstream area of the energy dissipator channel was strongly influenced by the discharge's magnitude and the quality of the river bed protection structures or riprap. The relative stability of the riverbed for Q6, Q7 (between Q_{1000yr} and Q_{PMF}), and Q8 (between Q_{1000yr} and Q_{PMF}) was 6 times, 10 times, and 20 times respectively higher from the condition at Q5. Meanwhile, for Q9 (equal with Q_{PMF}), the relative stability of the riverbed was considerably low.

3.4 Cross Flow Countermeasure

On almost all case of side channel spillway construction, the presence of cross flow on the chute channel always occur. The narrower the room on the downstream of side channel spillway, the higher the potential of the cross flow presence. The hydraulic impact of the cross flow occurrence is the increasing water level around the cross flow. Generally, it is difficult to eliminate cross flow, such as by constructing baffle block and or baffle wall on the upstream area of the spillway channel (see Figure 16). Based on the experiment, the baffle block and baffle wall construction did not give a significant effect on reducing or eliminating cross flow. Considering the increase in water surface elevation was also insignificant, then the effort to reduce or eliminate the cross flow by baffle block or baffle wall does not necessitate.



(a) Baffle block (b) Baffle wall

Figure 16 Baffle block and baffle wall impacts on crossflow behaviour

4.0 CONCLUSION

The side channel spillway can flow the Q_{PMF} and Q_{1000yr} with adequate freeboard. The water level height in the spillway downstream with Q_{1000yr} discharge reached an elevation of +352.88 m and elevation of +354.64 m for Q_{PMF}.

Considering the various discharges (between 0.006 to 0.010 m^3/s in the model or 600 to 1000 m^3/s in the prototype) that produced flows those exceeded the wall of the stilling basin wall, it is recommended to increase the height of the aforesaid wall from +212.50 m to +215.00 m.

Considering the scouring phenomenon on the downstream area of the stilling basin was a three-dimensional hydraulic phenomenon that interacted with the riverbed's condition which has certain erodibility, it is advised to keep a close observation on the scouring aspects in the downstream area of the stilling basin continuously. If necessary, several repairs to increase the riverbed's stability level from local scouring hazard could be applied, such as increasing the dimension of riprap materials for the bed protection structure.

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