Evaluation of Hydraulic Performance of Nazanin Dam Side Channel Spillway

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INTRODUCTION

Spillway is a hydraulic structure provided at the dams to release flood discharge and consequently to prevent over-topping. Side channel spillway has been commonly used with earth dams and dams that constructed in narrow gorges. Flow in this type of spillway is SVF with increasing discharge.

Side channel spillway includes six parts namely; side weir, trough channel, control section channel, transition section channel, chute channel and stilling basin. The simplest form of the side channel spillway is to have a straight weir perpendicular to the axes of the dam (Etheridge, 1996 and Knight 1989). Sometimes L and U shapes side weir are used in order to reduce the cost of excavation. Physical model is an essential tool for constructing model in the laboratory with a suitable scale ratio to show complex flow features, hydraulic performance and ensuring safe operation of the structure. SVF was...
initially investigated by Hinds (1926) who introduced the general equation of SVF with excluding friction loss. He proposed the concept of equivalent critical depth channel to determine the location of critical control point. Hager (1985) analyzed flow in trapezoidal side channel spillway and derived relations to determine the location of singular point for trapezoidal channel with moderate bottom slope. Bremen and Hager (1989) experimentally investigated flow characteristics in rectangular side channel spillway. They showed that due to the effect of jet of water over the weir the axial flow depth at side wall opposite to the side weir is greater than the flow depth at the centerline. Discharge coefficient of side channel spillway is different from normal spillway; it reduced as the discharge increased (Mandegaran, 1998). To ensure that the critical depth occurs within the channel length both dimensionless parameters (design capacity $F_q/S_o$ and channel roughness $N/S_o$) should be less than unity (Guo, 1999). (Machajski and Olearczyk, 2011) used physical model with 1:40 scale ratio to investigate hydraulic performance of side channel spillway. They observed that discharge coefficient reduced as side weir submerged and suggested deepening of the channel to improve its performance. For side channel with mild slope and vertical drop face side weir two vortex spiral flow observed at low discharges while a single vortex created at high discharges. This causes increase of water level in the channel at opposite side of the weir (Lucas et al., 2015). There is a limited experimental published research in this field. The present study deals with the evaluation of hydraulic performance of Nazanin dam side channel spillway. A physical model with scale ratio 1:40 was constructed to represent Nazanin dam side channel spillway at the hydraulic laboratory of Dams and Water Resources Engineering Department - College of Engineering at University of Salahaddin. Nazanin dam is a rockfill dam located at 71 km north east of Erbil city in Kurdistan region at the coordinate (459088 – 4010378) WGS_1984_UTM_Zone 38 N (Figure 1). The characteristics of Nazanin dam and the spillway are shown in Table 1.

Figure 1 Location of Nazanin Dam project.

### Table 1 Characteristics of Nazanin dam and the spillway

| Dam and Reservoir Characteristics | Type        | Rock fill                   |
|----------------------------------|-------------|-----------------------------|
| Crest Elevation                  | 730.2 m.a.s.l |
| B.L. of River                    | 709.2 m.a.s.l |
| M.W.L                            | 725.84 m.a.s.l |
| dam Height                       | 21 m        |
| Crest width                      | 7 m         |
| Total storage                    | 1864760 m3  |
| Spillway                         | Type        |
| Design discharge                 | S.C.S       |
| Trough channel                   | Length      |
|                                 | 67.65 m     |
|                                 | Width       |
|                                 | 29.65 m     |
| Side weir                        | Type        |
| Vertical Drop                    | Length      |
|                                 | 81 m        |
|                                 | Width       |
|                                 | 0.5 m       |
| Control channel                  | Length      |
| Transition channel               | 10 m - 18.71 m |
|                                 | Width       |
|                                 | 25 m        |
| Chute channel                    | Length      |
|                                 | 100 m       |
|                                 | Width       |
|                                 | 15 m        |
|                                 | Slope       |
|                                 | 8.4 %       |
| Stilling basin                   | Type        |
| Type II                          | Length, width |
|                                 | 20          |

### MATERIALS AND METHODS

#### Physical Model

The physical model for the present study with scale ratio 1:40 was constructed in the hydraulic laboratory of Dams and Water Resources Engineering Department (Figure 2a). The model constructed from high density
plastic Perspex sheets (acrylic). According to the recommendation due to Chanson 2004, for scale ratio 1:40 the roughness of plastic is almost equal to the roughness of concrete in the prototype. A large reservoir tank constructed upstream of the model to permit a steady water to flow over the model of spillway. The side weir consists of three segments, like L shape water comes into the trough channel at the upstream and its left side (Figure 2b). The physical model is similar to the prototype in terms of geometric, kinematic and dynamic. The gravitational force in modeling of the free surface flow is dominant. Therefore, Froude criterion of similitude is applied to calculate all the forces which occur in the model and the prototype. The Froude number of both the model and the prototype should be the same (Durgaiah, 2002, p.636). Froude’s law of similitude is used in modeling spillways, weirs, and sluice gates and in general for all free surface flow when the effect of viscous and surface tension forces are relatively small which can be neglected. The scale ratio is the ratio between model and prototype dimensions. The relations derived from Froude’s law of similitude as shown in table 2. Flow depths were measured using point gauge with the accuracy 0.05 mm. all measurements were taken when steady state achieved. The steady state condition achieved where the flow depth over V notch weir, side weir and all parts of the spillway does not changed with respect to time.

### Table 2 Froude law relationships for similitude

| Parameter | Ratio |
|-----------|-------|
| Length    | $L_r$ |
| Velocity  | $V_r$ |
| Area      | $A_r$ |
| Discharge | $Q_r$ |

Hydraulics of Side Channel

When lateral inflow enters the side channel, it causes a considerable energy loss in the channel, due to turbulent mixing of water that enters the channel and water flowing in the channel (French, 1985). Flow in side channel is more complex due to variation of discharge along its length. Therefore, momentum equation is used to develop dynamic equation of spatially varied flow (Chow, 1959).

### A. Dynamic Equation of SVF With Increasing Discharge

The equation of SVF with increasing discharge can be obtained based on the assumption of conservation of linear momentum. Subramanya (2009) suggested the following assumptions to apply momentum to the control volume:

1. The streamlines are parallel, hydrostatic pressure distribution prevail with excluding locations of highly curvature in the analysis.
2. The momentum correction factor is used to represent the effect of non-uniformity of velocity distribution.
3. Manning formula used to estimate the friction losses.
4. The effect of air entrainment on forces involved in the momentum equation is neglected.
5. The inflow rate is constant and it does not contribute to any momentum in the streamwise direction.
6. The flow is steady and channel is prismatic with small slope.

The dynamic equation of SVF with increasing discharge as presented in (Chow, 1959) is:

\[ \frac{dy}{dx} = S_o - S_f - \left( \frac{2 \beta Q q}{g A^2} \right) \frac{1}{1 - \beta \frac{Q^2 T}{A^3 g}} \]  

Where: \( S_o \) is the longitudinal bed slope of the trough channel, \( S_f \) is the frictional slope, \( q \) is the lateral inflow \( (m^2/s) \), \( Q \) is the total discharge \( (m^3/s) \), \( A \) is the cross sectional area of the trough channel \( (m^2) \), \( T \) is the flow top width \( (m) \), \( g \) is the gravitational acceleration \( (m/s^2) \), \( \beta \) is the momentum correction factor, \( x \) is the horizontal distance along the channel and \( y \) is the depth of flow.

The above equation is hardly to be integrated. Therefore, to draw the water surface profile in the side channel the numerical integration form using trial and error technique can be used as presented in (Chow, 1959).

\[ \Delta y = \alpha Q_1 (V_1 + V_2) \left( \Delta V + \frac{V_2}{Q_1} \Delta Q \right) g (Q_1 + Q_2) + S_f \Delta x \]  

\[ x_t = \frac{2}{(S_o T - g A^2)} \frac{Q_o}{q} \]  

\[ x_c = \frac{1}{q} \sqrt{\frac{g A^3}{\beta T}} \]

Where: \( x_t \) is the distance to the transitional depth \( (m) \), \( Q_o \) is the discharge at the upstream of the trough channel \( (m^3/s) \), \( K \) is the conveyance factor, \( x_c \) is the distance to the critical depth \( (m) \).

The intersection point between transitional profile with the critical depth line is the critical control depth.

**Discharge Equation**

The discharge over the side weir of the spillway can be calculated using the equation of sharp crested weir adopted by Subramanya (2009) as follows:

\[ Q = \frac{2}{3} C_d \sqrt{2g L h_o^{3/2}} \]

Where: \( C_d \) is the discharge coefficient, \( L \) is the length of the weir, \( h_o \) is the water depth over the weir.

At most discharges the weir acts as a sharp crested weir.

**RESULTS AND DISCUSSION**

**Water Surface Profile (WSP)**

The WSP was obtained from the physical model by measuring flow depth at
various sections along the spillway including centerline, right and left side walls for eight discharges. In the trough channel the transitional profile and critical depth line did not intersect each other (Figure 3) for this reason the critical control point was fixed at the end of the control channel. Furthermore, the WSP was computed in the trough channel using equation (2).

![Figure 3 Critical and transitional profile in trough channel for different discharges.](image)

The results obtained from the physical model and calculated WSP are outlined in the following parts:

**Trough Channel**

In the trough channel the computed, design report and observed WSP are very close to each other (Figure 4). However, the WSP for the design report is only drawn for the maximum discharge as it represents the design discharge on which the structure has been designed accordingly. This reveals that equation (2) adequately predicted WSP. Only there are some deviations between them at the beginning of the channel, which might be due to falling jet of water over the weir caused reduction of flow depth. This phenomenon cannot be handled by one dimensional equation. All results indicated that flow in the trough channel is subcritical, Froude number was increased along the flow direction and approaches unity at the outlet (Table 3). Type of flow is type (A) according to the classifications made by Li 1955 (presented by Chow, 1959). Furthermore, at the design discharge (33.451 l/s) the side weir approaches to the submergence condition. In all discharges flow depth at the opposite of the side weir is greater than the flow depth at its centerline.

**Table 3 Calculated Froude number in the trough channel**

| Distance from US (m) | Froude Number |
|----------------------|---------------|
| 0.5                  | 0.25345       |
| 0.75                 | 0.30684       |
| 1.0                  | 0.36448       |
| 1.3                  | 0.42585       |
| 1.6                  | 0.62026       |
| 1.6952              | 0.72788       |

**Transition and Chute Channel**

Flow in the transition channel is highly non-uniform. Due to shape of the transition and contraction of width of the channel from 0.625 m to 0.375 m cross waves observed in the transition channel and traveled into the chute channel (Figure 5). Therefore, raising water level occurred near boundaries of the transition and chute channel. Furthermore, extreme overtopping observed at the transition and middle of the chute channel. While, at end of the chute channel intensity of waves reduced and raising water level diminished and the flow approximately become uniform. The results of
WSP observed from the physical model are higher than that presented in the design report at the transition and start of the chute channel. The side wall height of the chute channel should be increased from 2m to 2.5m (according to the prototype) to prevent overtopping.

**Stilling Basin**

The hydraulic jump observed at the outlet of the stilling basin this causes high overtopping at both sides. The result of sequent depth obtained from the physical model is less than that presented in the design report. This can be attributed to the fact that in the design report Blenger momentum equation for rectangular channels has been used to calculate the jump sequent depth, whereas the stilling basin of the structure isn't of rectangular shape. Due to the absent of the chute and baffle blocks, the flow for higher discharges was continuous until reaches end of the basin then the jump was formed (Figure 4a, b). For lower discharges a part of the hydraulic jump comes into the chute channel (Figure 4d) and causes overtopping at both sides of the chute channel.

**Discharge Coefficient**

The discharge coefficient for the present was calculated in two conditions:

1. Discharge coefficient when velocity head is neglected from the calculation. In spillways at lower value of the ratio of \((h_o/p)\) the velocity head is very small that cannot affect the results. The discharge coefficient increase as the discharge increase but at higher discharges the discharge coefficient decreases this related to the flow condition in the side channel when the
weir crest approaches to submergence condition (Figure 6a). The results showed that $C_d$ at the design discharge is 0.7228, while in the design report $C_d = 0.60975$, so the weir can pass extra discharge by 18%.

2. Discharge coefficient when velocity head is considered from the calculation. The results are shown in Figure 6 b, which seems the same behavior as in previous case. From both results it is evident that the velocity head is very small so that it can be neglected.

$$C_o = -0.6776(Q/Q_d)^2 + 1.0193(Q/Q_d) + 0.3835$$
$$R^2 = 0.9948$$

Figure 6 Discharge coefficient versus discharge without and with velocity head

Energy Dissipation

The percentage of energy dissipation is calculated using the following equations.

$$H_1 = h_o + \frac{V_o^2}{2g} \quad \text{......................... 6}$$

$$H_2 = y_2 + \alpha \frac{V_2^2}{2g} \quad \text{......................... 7}$$

% Energy Dissipation

$$\frac{H_1 - H_2}{H_1} \times 100 \quad \text{............ 8}$$

$H_1$ is the total upstream head, $H_2$ is the residual head at the downstream of the stilling basin, $V_o$ is the average velocity in the reservoir, $V_2$ is the average velocity at the downstream of the stilling basin, it was calculated using continuity equation (Q/A).

The energy correction factor ($\alpha$) is assumed be equal to unity, since the channel cross section is regular.

Figure 7 shows the relationship between percentage of the energy loss and the discharge. It is clear that the percentage of energy dissipation decreases as the discharge increases. Percentage of the energy dissipation is equal to 57.5% at the design discharge. This value is very close to the percentage of energy dissipation calculated from the design report which is equal to 56.6%. According to the value of the percentage energy dissipation it is clear that the stilling basin cannot dissipate energy sufficiently, so the river bed is likely prone to scouring. As mitigation measures against such damage, chute and baffle blocks should be added at the end of the stilling basin.

**CONCLUSION**

In the present study the hydraulic performance of Nazanin dam SC Spillway was evaluated. The physical model was used to determine water surface profile in the spillway.
for discharges ranging from (2.56 – 33.45) l/s. In addition, the discharge coefficient and the energy dissipation at the downstream of the stilling basin were studied. The results of analysis of data obtained from the physical model can be outlined in the following conclusions.

1. Effect of water jet over the weir on the water surface in trough channel at opposite side of the weir is small. Because the channel is wide and shallow.
2. It was found that the dynamic equation for SVF with increasing discharge equation (2) can be used to predict 1D water surface profile in trough channel for L shaped weir, since it gave reasonable results.
3. Flow is subcritical along the entire length of the trough channel according to the location of CCP; this was approved by the physical model results.
4. The shape of the alignment and contraction width of the channel are main factors for generating cross waves in transition and chute channels.
5. The discharge coefficient is very sensitive to the flow condition in trough channel. Its value decreased when the weir approaches to the submergence.
6. The percentage of energy dissipation varied inversely with the discharge.
7. The stilling basin is hydraulically inefficient, it needs extra elements to dissipate energy of flowing water.

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