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Experimental Investigation of the Original Project of Yukari Kaleköy Dam and Hydroelectric Power Plant Spillway

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Abstract

Hydraulic model tests of the spillway of Yukarı Kaleköy Dam and Hydroelectric Power Plant (HPP) were carried out at Hydromechanics Laboratory of Middle East Technical University (METU), Ankara. In order to investigate the flow conditions in the approach channel, over the weir structure, around the spillway piers, along the spillway chute channels and after the flip buckets, and also to observe the performance of the aeration devices, a model of 1/70 length scale was constructed. The model area covers an about 400 m × 450 m section of the Yukarı Kaleköy reservoir, the whole spillway structure and a part of the river area of about 150 m × 200 m downstream of the flip buckets. Experiments were carried out at various discharges; QPMF, Q_{1000} and Q_{100} with fully open radial gates as well as with partly open radial gates at Q_{1000}. Flow depths, pressure heads of the flows along the spillway structure were measured, cavitation indexes and air concentrations of the flows were determined. From the observations and experimental results, it was concluded that the flow conditions in the approach channel of the spillway, over the weir structure, around the spillway piers, along and after the flip buckets are quite good from the hydraulics point of view. However, the flows were not sufficiently aerated in the spillway chutes, in Channels I and II, after the available aerators presented in the original design of the spillway. After the experiments conducted on the "original model" it was decided that some modifications should be done. In order to improve the performance of the available aerators, the heights of the aerator deflectors were increased by 0.20 m by keeping their lengths in the flow direction almost as before.

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The same experiments which had been conducted for the “original model” were repeated on this “modified model” too. Consequently, on the modified model it was shown that the flows after the spillway piers and aerators are almost uniformly distributed in each spillway chute and sufficiently aerated without having cavitation risk on the structure.

1. Introduction

The Yukarı Kaleköy Dam and HPP is to be constructed on the Murat river within the boundaries of Bingöl province. The main function of the Yukarı Kaleköy Dam and HPP is to produce power. The installed capacity of the project is proposed to be 630 MW from 3+1 Francis turbines. The weir type is concrete with four radial gates and located on the left bank of the dam. The spillway has been designed with respect to the probable maximum flood discharge of Q_{PMF}=8476.4 m^3/s at reservoir water level (RWL) of 1237.55 m. After the weir structure, the spillway chute is divided into two channels; Channel-I on the right and Channel-II on the left in the flow direction, by a separation wall and located according to the topography of the site. Top view of the spillway weir and chutes at Q_{PMF} is shown in Fig. 1.

Figure 1. Top view of the model.

According to the agreement the METU Hydraulics Laboratory undertook the following studies stated by Kalehan Enerji Üretim A.Ş. as the model requirements, [3].
Investigation of approach flow situation and headwall design; investigation of the weir overflow conditions and eventually optimization of the design of the weir and the piers (shape and location of bridge nose); investigation of the weir flow for several discharges and gate positions; investigation of the air intrusion and cross-waves due to channel contraction and pier end; and determination of freeboard; investigation of the form and location of aeration devices; and investigation of trajectory distance for several discharges and if trajectories are found problematic, optimization of flip bucket.

2. Experimental Studies

In order to simulate the flow conditions as closely as possible a large portion of the reservoir, the weir structure and chute channels of the spillway, and some part of the downstream river after the flip buckets were included in the construction of the model. The water is provided from the main storage of the laboratory by an intake pipe to the storage zone located to the upstream of the model. This zone where the energy of the flow is reduced, separated by a wall formed by bricks of small rectangular openings from the reservoir of the dam. By means of this brick wall stabilized and turbulence free flow is directed towards the spillway structure. The discharges of the model used in the experiments were measured by an acoustic flow meter mounted on the intake pipe of the model. In the measurements of; flow depths a point gauge, velocities a mini-propeller meter and pressure heads piezometer tubes connected to piezometer openings were used. Once the model construction was completed, an experimental program has been initiated systematically. After observing that the spillway is capable of passing the possible maximum flood discharge of Q_{PMF}=8476.4 m^3/s determined from flood routing by [2] at RWL=1237.55 m, important structural components of the model were tested to see whether or not they function properly from the hydraulics point of view. Initially, the experiments were performed on those components of the original design and the results obtained are presented in the following section, [1].

3. Results and Discussions

3.1 For the Original Model

- The lengths and forms of the side walls of the approach channel are sufficient to guide the incoming flow almost uniformly towards the spillway without causing undesired flow conditions in the approach channel.
- At the maximum discharge, Q_{PMF} =8476.4 m^3/s, strong flow separations and depression on the water surface are observed around the curved upstream section of the right side wall of the spillway approach channel. However, these irregularities occurring on the free surface disappear as the flow approaches to the weir structure.
- The flow conditions around the spillway piers are quite good for all the discharges tested; water surfaces entering the spillway bays are sufficiently smooth and result in almost uniform flows over the weir structure.
- Water filling the gate slots has a rotating nature starting from the free surface and penetrating down towards the channel bottoms at each discharge tested.
- The measured spillway capacity of the model at the reservoir water level of 1237.55 m is 8470 m^3/s and almost the same as Q_{PMF} used in the design of the structure.
- The correlation between the theoretical and measured discharge capacity of the spillway is quite good for fully and partially open gate conditions.
- Under different RWLs each spillway bay carries almost the same amount of discharge.
- Due to the contracting nature of the chute channels, in general, water depths along the side walls of the channels are larger than those of the channel centrelines except a few sections.
- The available wall heights of the chute channels are adequate not to have water spill over the walls along the channels with one exceptional case of Q= Q_{PMF} /2=4238.2 m^3/s, at which the depth of the flows exceeds the wall heights at two sections in Channel-I and Channel-II.
- In the zone of the flip buckets in Channel-II, the depth of the flow exceeds the available wall height along the wall separating the two channels by about 8 m. However, the flow jets leaving the flip buckets do not disturb each other significantly.
- The lengths of the flow jets leaving the flip buckets in each spillway chute channel are compatible with those corresponding values determined theoretically by TEMELSU (2013).
- The first two aerators in Channels-I and II cannot fully aerate flow at Q=Q_{PMF}/2=4238.2 m^3/s.
- The minimum pressure head measured along the spillway chute channels is -0.98m in Channel-I for Q=Q_{1000}/2=2183 m^3/s. This section and the others at which negative pressure heads are measured are generally located just after the aerators within the flow jet lengths.
- The cavitation index, σ, has the values less than 0.2 at a few points in both spillway chute channels at discharges tested. The minimum values of σ are 0.163 and 0.174 in Channel-I and Channel-II, respectively, for Q= Q_{PMF} /2=4238.2 m^3/s.

- The calculated average air velocities in the air vents of Channels-I and II are less than 100 m/s.
- The minimum average air concentrations of the flows are 0.0945 and 0.0774 in Channels-I and II, respectively.

After completing the experiments on the hydraulic model constructed according to the original design and analysing the experimental results with the authorities of [2], it was concluded that there was no need to modify the approach channel, its side walls and the shapes and dimensions of the spillway piers upstream and downstream of the weir structure. The most important observation made during the experiments was the performance of the aerators some of which were not aerating the flow property. The measured flow jet lengths after the aerator ramps were not long enough to provide sufficient air into the flow. To eliminate this undesired situation, it was decided that the deflector heights of the aerators in both channels; Channels-I and II, should have been increased by 0.20 m. Therefore, the deflector heights of each aerator were increased from 0.40 m to 0.60 m by keeping the initial deflector length almost the same. Figures 2 and 3 show the dimensions of the aerators in the original and modified designs, respectively.
All of the tests conducted on the model of the original design were repeated on the modified hydraulic model for the discharges of $Q=Q_{PMF}=8476.4$ m^3/s, $Q_{1000}=4362$ m^3/s and $Q_{100}=3256$ m^3/s and were summarized in the following sections [1].

3.2 For the Modified Model

- In the modified model the flow conditions in the approach channel of the spillway over the weir structure, around the spillway piers and along the chute channels until the first aerators are the same as those of the "original model".
- Only at $Q=Q_{PMF}/2=4238.2$ m^3/s the flow depth in Channel-II exceeds the wall height at around 160 m from the crest by about 1 m. As discharge decreases, no spill of water over the spillway chute walls is observed.

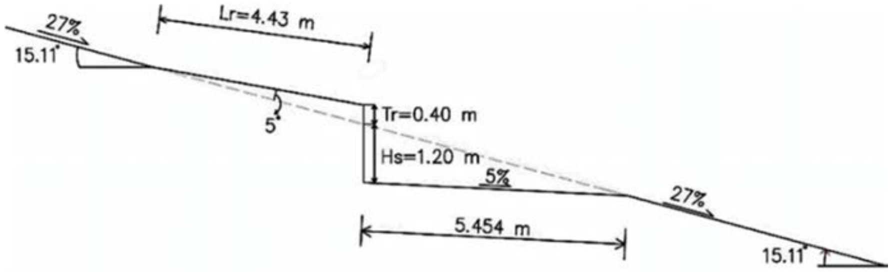

Figure 2 Dimensions of the aerators in the original model.

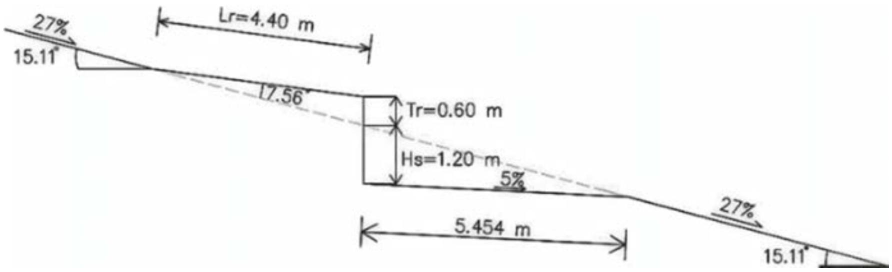

Figure 3 Dimensions of the aerators in the modified model.

- The flow conditions in the zones of the flip buckets and the lengths of the flow jets leaving the flip buckets are very similar to those of the "original model".

Due to the increased deflector heights of the aerators the flows are fully aerated in the "modified model" after each aerator in both Channels-I and II.

- The minimum pressure head measured along the spillway chute channels is -0.42 m in Channel-I for $Q=Q_{1000}/2=2181$ m^3/s.

The number of locations on the spillway chute channels where the cavitation index σ is less than 0.2 is just a few in the "modified model" with the value of 0.177. Since the flows are fully aerated at various discharges tested, it can be concluded that at these locations where σ values are less than 0.2, there will be no cavitation risk.
The calculated average air velocities in the air vents of Channels-I and II are less than 100 m/s at most of the aerators except 3rd and 4th.
The minimum average air concentrations of the flows are 0.1210 and 0.1114 in Channel-I and Channel-II, respectively.

- From the experiments conducted on the "modified model" for $Q=Q_{1000}$=4362.0 m^3/s with partially open gates at RWL= 1235.00 m the following conclusions can be stated:

The flow conditions over the whole spillway structure are quite good and can be considered as acceptable from the hydraulics point of view.

- The measured maximum negative pressure heads occur just downstream of the aerators with the values of -0.42 m in Channel-I and -0.07 m in Channel-II.

The minimum cavitation indexes determined are 0.109 and 0.131 in Channel-I, and 0.186 and 0.182 in Channel-II. Since at these sections the average air concentrations of the flows are 0.1660 and 0.1370 for Channel-I and Channel-II, respectively, there will be no cavitation risk.
The calculated average air velocities in the air vents of Channels-I and II are less than 100 m/s.

- The minimum average air concentrations of flows are 0.1498 and 0.1370 in Channel-I and Channel-II, respectively.

4. Conclusions

The following recommendations can be made regarding the results of the experiments conducted on the original and modified models.

- There is no need to make changes on the original design of the approach channel, the lengths and shapes of the approach channel side walls, the weir structure, dimensions and shapes of the spillway piers and the flip buckets of the chute channels.
- The gate slots through which vortices form should be lined with proper metal sheets to protect their inner surfaces from the adverse effects of the vortices.
- The heights of the original aeration deflectors should be increased by 0.20 cm while the offset heights of the aerators are being kept as proposed in the original design (Figures 2 and 3).
- The heights of the walls of the spillway chute channels can be kept as proposed in the original design. Only, at the flow rate of Q_{PMF}, at just a few sections of the spillway chute channels, water spills over the walls and drop either into the next channel or outside the channel without disturbing the flow conditions in the chute channels.
- The depths of the flows occurring along the flip buckets exceed the height of the horizontal wall separating the two flip buckets but actually this situation does not show any adverse effect on the flow jets leaving the flip buckets. Therefore, if it is desired, the height of the separation wall can be increased by 8 m.

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References

[1] Göğüş, M., Altan-Sakarya, A.B., Köken, M., Yavuz, C., Dinçer, A.E., 2014. Yukarı Kaleköy dam and hydroelectric power plant spillway hydraulic model studies. Final Report. Middle East Technical University, Ankara, Turkey.
[2] Temelsu, 2013. Upper Kaleköy Dam and Hydroelectric Power Plant, Hydraulic Calculations.
[3] Knauss, J. 1987. Swirling Flow Problems at Intakes, A.A. Balkema, Rotterdam.