Optimization by physical and numerical modelling of the spillway for the Spullersee dam rehabilitation project

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Introduction
The "Spullersee" hydropower scheme (Fig. 1) was built from 1922 to 1926 in the Vorarlberg region (Austria) in order to cover the increasing electricity demand of the Austrian railroads (owner). The total catchment area with additional contributions is 18.4 km². Two gravity structures of 24 and 35 m height formed the original reservoir with a normal water level at 1825 m a.s.l.. In 1965 both dams have been heightened allowing a 4.6 m rising of the normal water level. The available storage capacity reaches now 15.7 × 10⁶ m³. The power station, located in Wald at 1015 m a.s.l. provides a power of 36 MW producing 48 GWh yearly.

The heightening project included the pouring of 3.3 m high concrete blocks on the crests of the dams and the installation of vertical pre-stressed anchors. The BBRV-type anchors had been grouted on the entire length, a common practice at that time, and therefore no reliable control of their efficiency or of the corrosion protection is possible any more.

The lack of reliable control methods combined with some signs of corrosion does no longer allow considering the contribution of the pre-stressed anchors to the stability of the dams. Remedial measures have thus been developed to provide adequate stability conditions of the dams. The proposed solution consists of replacing the stability contribution of the anchors with a rockfill shoulder placed on the downstream faces of the dams.

Since they are located at the centre of the dams, two of the three existing overflow spillways need to be abandoned within the rehabilitation works. Therefore, the capacity of the main spillway on the right abutment of the south dam had to be increased from 25 m³/s to 74 m³/s. The future construction is formed by a 57 m long U-shaped crest, followed by a rectangular trough and a 62 m long curved chute (kind of duckbill spillway).

Numerical calculations as well as experiments on a physical model (scale 1:15) have been done for the verification and optimization of the spillway design. The numerical Flow-3D calculations are generally in good agreement with the physical model experiments. Nevertheless, some particular flow structures and characteristics have been detected only on the physical model.

The article shows the new flood evacuation concept, a comparison of the results for the new spillway between numerical calculations and physical experiments and finally the realization of the construction of the spillway.

Fig. 1: Left side: Photo of the existing chute spillway. Right side: General layout of the Spullersee high head hydropower scheme (Bremen et al. 2004)
1. Previous and future flood evacuation concept

Prior to the rehabilitation works, the plant was equipped with tree spillways. The main spillway on the right bank of the south dam consists in a 20 m long circular sill at el. 1829.6 m a.s.l. having a maximum capacity of 25 m$^3$/s. Two auxiliary overflow spillways with crest at el. 1830.0 m a.s.l. were located at the centre of the dams. The 102.7 m long spillway on the south dam reached the maximum capacity of 42 m$^3$/s while the 16.9 m long on the north dam only 7 m$^3$/s. It must be mentioned that since the heightening in 1965 the two auxiliary spillways never discharged. The south dam is furthermore equipped with a 15 m$^3$/s capacity bottom outlet, which however can’t be taken into account for the flood evacuation.

The direct drainage basin area is 11.10 km$^2$. Four additional catchments increase the total drainage basin area to 18.4 km$^2$. The hydraulic design considers a 5000-year flood with an inflow of 96 m$^3$/s. With an initial reservoir level at el. 1829.6 the flow damping of the reservoir is about 22 m$^3$/s, resulting in a peak outflow discharge of 74 m$^3$/s by a maximum impounding level at el. 1830.4 m a.s.l.

The strengthening of the dams with a rockfill shoulder placed on the downstream faces required an adaptation of the flood evacuation concept, as shown in Fig. 2. New parapets close the two auxiliary overflow spillways located at the centre of the dams and the discharge capacity of the main spillway was increased from 25 to 74 m$^3$/s. The maximum discharge on the south dam increased of 7 m$^3$/s in comparison to the previous layout.

A 57 m long crest, followed by a rectangular trough and a 62 m long chute as shown in Fig. 3, forms the new spillway. The existing chute spillway could be integrated into the new construction. The horizontal orientation of the trough follows the location of an existing flat surface, which could be used for the foundation simplifying significantly the constructions works.
In order to verify the adapted spillway, detailed and extensive numerical analyses (Hydrocosmos SA, 2001) have been initially proposed with the purpose to avoid laboratory studies. The Control Authorities required however a study on a physical model (LCH, 2002) which allowed a further optimisation of the new spillway.

2. Physical and numerical modeling

Generally known formulas have been used for the initial design of the spillway by the engineering company (Lombardi Engineering Ltd.). It was assumed that by the use of calculation reserves as well as careful hydraulic assumptions, detailed 3D-computations by Flow3D (Hydrocosmos SA) would be sufficient. Nevertheless, at the time of the project evaluation by the Austrian commission on reservoirs, it was decided to proceed with hydraulic model tests, which have been done at the Laboratory of Hydraulic Constructions (LCH) of the Swiss Federal Institute of Technology Lausanne (EPFL).

For the new spillway of the Spullersee reservoir, results of physical test as well numerical calculations are thereby available. They are presented and compared in the following chapters.

2.1 Physical model

The physical model was built at the geometric scale 1:15 (Fig. 4). This scale allows excluding scale effects due to the water viscosity of the water. The basin, in which the spillway trough and the topography of the lake are integrated, has a surface of 40m×60m (all dimensions are prototype values). This size allows a correct reproduction of the flow conditions next to the planned spillway. The curved chute is embedded outside of the basin and its length is 62 m.

The objectives of the experiments on the physical model at scale 1:15 were:
- Verification of the 74 m$^3$/s outflow capacity at the maximum water-level of 1830.39 m a.s.l.
- Measurement of the flow-levels in the trough and along the sidewalls of the curved chute
- Choc wave analysis on the curved chute
- Hydraulic optimisation of the structure geometry
- Validation of the 3D-computations

The following measurements have been done to describe the relevant parameters:
- Discharge of the water supply
- Water level in the lake, in the trough and along the sidewalls of the chute
- Flow patterns at the water surface in the lake by taking photographs of floating candles
- Flow velocity at the end of the trough by micro-propeller

The physical model experiments have been done for three main alternatives of the trough. Alternative A has a rectangular plan view of the trough (Fig. 4). Alternative B is similar, but with an hexagonal upstream end of the trough (Fig. 4). Alternative C is similar to Alternative B, but with a bottom of the trough lifted up for 46 cm. Alternative B was proposed with the purpose to reduce some important flow disturbances on the right side of the overfall with air entrainment by vortices (see Fig. 4, middle and right side). Alternative C was tested during the construction of the trough with the aim to verify the capacity with a lifted up bottom in order to reduce the in site rock excavation. The curved chute was never modified during the experimental phase.

![Fig. 4: Left side: Physical model at scale 1:15 of the new flood evacuation structure with the basin, trough and curved chute. Middle: Alternative A for the trough (Q=74 m$^3$/s). Right side: Alternative B for the trough (Q=74 m$^3$/s)](image-url)
2.2 Numerical model

The numerical modeling of the modified spillway for the Spullersee dam was achieved with the software FLOW-3D, developed by Flow Science Inc. in Los Alamos. The numerical code solves the Navier-Stokes equations in three dimensional space and allows so the simulation of 3D transient flows.

The geometrical model was processed by Lombardi Engineering Ltd using the software Autocad. The data issued from this program can be directly interpreted by FLOW-3D.

The surface roughness of the hydraulic works was admitted as 1.5 mm, corresponding to a standard concrete execution.

The upstream boundary conditions are defined as a constant water head corresponding to the maximum reservoir level at el. 1830.4 m a.s.l.. The hydrostatic pressure distribution is admitted constant in the reservoir. The downstream boundary conditions correspond to a free surface flow.

2.3 Results

As the numerical calculations were done for Alternative A, most of the following results concern this geometry.

The results are presented respecting the direction of the flow: a) Water level in the lake for different discharges b) Flow conditions next to the overfall-crest c) Water-levels in the trough d) Flow conditions on the chute.

a) Water level in the lake for different discharges and geometries

The rating curves of the crest overfall have been measured for the two alternatives. In order to compare the measured values with a theoretical approach, calculations have been done as proposed by Hager (1992) for cylindrical overfall crests. Relation between discharge (Q) and waterhead level (H) is done by:

\[ Q = C_d \cdot L \cdot \sqrt{2g \cdot (H - 1829.60)^{1.5}} \]  

(1)

The overfall-coefficient \( C_d \) can be calculated by:

\[ C_d = 0.374 \cdot (1 + \frac{3\rho_k}{11 + 2.5\rho_k}) \quad \text{with} \quad \rho_k = \frac{H}{R_k} \]  

(2)

\( R_k \) is the radius of the cylindrical overfall crest (\( R_k = 0.45 \) m). \( L \) is the length of the crest (\( L_A = 53.22 \) m; \( L_B = 56.54 \) m). This geometric length is reduced of about 0.18 m for a discharge of 74 m\(^3\)/s by contraction of the flow at the beam (Fig. 4).

The results of measurements and calculations are presented in Fig. 5 above. Alternative A can achieve the capacity of 74 m\(^3\)/s for a water-level exactly equal to the maximum 1830.39 m a.s.l.. Alternative B reaches the capacity of 74 m\(^3\)/s for a water-level of 1830.36 m a.s.l.. The difference between Alternative A and B is due to the increase of the crest length. The difference between the theoretical and measured rating-curves can be
explained by 3 observations on the physical model: First point is the reduction of the effective length at the corners of 90° (Alternative A) respectively 120° and 150° (Alternative B) at the upstream end of the trough. Second point is the disturbance of the flow conditions at the shoreside crest. Third point is the reduced water depth on the shoreside due to the topography which lead to higher velocities and non perpendicular streamlines to the crest. The influence of this third point can be expressed by a reduction of the $C_d$-coefficient. Separate tests have been done for the sea- and shoreside crests in order to determine appropriate $C_d$-coefficients. While the experimental seaside $C_d$-coefficient is about constant for all energy levels ($C_d = 0.498$ to 0.494), the shoreside $C_d$-coefficient is reduced from 0.485 ($H=0.72 \, m$) to 0.420 ($H=1.16 \, m$). It can be concluded that the three mentioned points (corners, flow conditions and topography) lead to a reduction of the effective overfall length of 10.3% for Alternative A and 8.6% for Alternative B.

In comparison, the numerical computations for Alternative A led to a discharge of 74 m$^3$/s for a water level of 1830.37 m a.s.l. in the lake. This result is in very good agreement with the measures on the physical model, which lead to 1830.39 m a.s.l. for the same discharge.

b) Flow conditions next to the overfall-crest

Generally good flow conditions are observed in the approach and along the structure. Only the flow over the shoreside crest of the spillway is disturbed under the influence of the topography and the shape of the trough (Fig. 4). The proposed geometry of Alternative B (Fig. 4, right) leads to a small reduction of the disturbances, but without eliminating them. The phenomenon appearing like a stationary wave is due to the detachment of the flow at the corners of the trough at higher velocities and non perpendicular flow lines referring to the crest on the shoreside. As well the measured as the computed velocities at the water surface in the lake are increased of 50 to 100% comparing to the seaside. Velocities are about 0.19 m/s to 0.34 m/s at the seaside and 0.29 m/s to 0.61 m/s at the shoreside. However, it was only possible to become aware of these irregularities by the observations on the physical model.

c) Waterlevels in the trough

The measured water depths in different sections of the trough are varying from 2.72 m upstream to 1.77 m at the beginning of the curved chute. Comparing to the height between the crest and the bottom of the trough, its filling
degree varies from 63% to 38.5%. The loading of the trough can thereby be excluded which can also be seen on Fig. 5 where the measured rating curves do not suffer of tailwater influence until at least 110 m$^3$/s. Thanks to this reserve, it was possible to increase the bottom level of the trough during construction of 0.46 m without reducing the capacity of the new spillway.

A comparison between the measured and computed water surface is done in Fig. 6 on the previous page for two cross sections (in the upstream part of the trough and at the beginning of the curved chute) and for a longitudinal section (along the axis of the trough). The agreement between the computed and measured water surface is particularly good in the second part of the trough. In the upstream part, the agreement is convenient as well, even if the flow levels next to the axis of the trough are slightly overestimated. Maximum difference between computed and measured water surface is about 0.5 m.

It can be summarized that Flow-3D is able to reproduce correctly the flow depths at the overfall crest, the general aspect of the water surface in the trough, the flow levels in the downstream part of the trough and the inclination of the water surface in the cross section at the beginning of the curved chute. Nevertheless, local flow phenomenon like vortex formation on the overfall crest could not be detected by Flow-3D in this case.

d) Flow conditions on the chute

The flow conditions on the chute are similar for the physical and numerical models (Fig. 7). A raising of the water level along the right wall and shock waves can be observed on the physical model. These two phenomena are qualitatively well reproduced by the numerical computations. As an example, the sudden water depth elevation at the end of the chute on the left wall due to the reflected shock wave from the right wall can be clearly observed in both experimental and numerical simulations.

Flow depths on the curved chute are varying with time as a result of shock waves. Therefore, mean and maximum flow depths have been measured along the left and the right chute-walls. The comparison of the mean flow depths measured on the model and the Flow 3D-computations shows that the numerical flow depths are slightly underestimated by max. 0.3 m. This effect may result from the air entrainment on the chute which could be observed on the physical model. It is also visible that the flow is more concentrated to the left wall in the experiments than in computations (Fig. 7).

![Fig 7: Comparison of the flow conditions on the curved chute for the design discharge $Q=74$ m$^3$/s. Left: FLOW-3D prototype computed velocity field and water surface with elevation lines every 0.5 m. Right: flow pattern on the physical model.](image-url)
3. Spillway construction

The rehabilitation works started in Spring 2002 with the demolition of the old spillway. The reservoir was temporarily lowered during this period. At the end of August the construction of the new spillway was practically finished, allowing the rest of the works to be done in security from floods.

Regarding the design some minor modifications occurred during the construction, as the fact that the floor of the new spillway trough has been raised of 46 cm in order to limit the quantities of rock to be excavated.

Anchors (Steel S500, l=2.8m, 1Ø20mm/4m²) and drainage holes (1Ø80mm/6.5m²) provide adequate stability conditions, in the vertical direction, of the trough. Fig. 8 shows the steel reinforcement and the drainage pipe before the pouring of the concrete floor of the trough.

The works carried out in 2002 included as well the elevation of the chute walls and the preparation of the foundation for the rockfill of the south dam. The nearly 49 m long walls of the chute spillway are 3.2 m high on the left side and 3.7 m on the right.

During 2003 the program continued mainly with the construction of the new access and drainage adits at the downstream dam toe and in 2004 with the construction of the embankments. Now, after a relative long interruption during the last winter, finishing works are underway allowing to complete the full rehabilitation of the Spullersee dams during the current year.

![Fig. 8: Left side: View of the new spillway trough during installation of the reinforcement and the drainage pipes in the floor. Right side: Completed spillway](image)

4. Conclusions

The rehabilitation project of the Spullersee dam in Austria led to abandon of two of the three existing overflow spillways. Therefore, the remaining spillway had to be modified in order to increase its capacity from 25 m³/s to 74 m³/s. The projected structure, formed by a 57 m long crest followed by a rectangular trough and a 62 m long curved chute, was submitted to numerical and experimental simulations with the purpose of validation and optimization.

The comparison of the results issued from Flow3D numerical computations and experimental tests achieved on a 1:15 scaled model shows good agreement. Nevertheless, some differences could be put in evidence due to particular phenomena like vortices, stationary waves and air entrainment. The most important gap between computed and measured values amounts to 50 cm for the water level in the upstream part of the trough and to 30 cm along the chute when considering the design flood of 74 m³/s.

The results obtained first from the numerical modelling and later on the physical model allowed to verify the hydraulic behaviour and to optimize the design of the spillway. Main changes brought to the initial geometry concern the upstream form of the weir crest, the increase of the bottom level of the trough and the heightening of the lateral walls of the chute.

The civil engineering activities of the Spullersee dams started in 2002. Finishing works are underway to complete the full rehabilitation during the current year.
References


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