

Dam flood routing capability: an example of reservoir discharge facilities rating curves determination through dedicated CFD simulations.

Simone Citterio
Frosio Next S.r.l.
Via Corfù, 71
25124 Brescia (BS)
Italy

Luigi L. Papetti
Frosio Next S.r.l.
Via Corfù, 71
25124 Brescia (BS)
Italy

Introduction

Dams, with their fundamental function for water management, have a crucial role in flood negative impacts mitigation. Hydropower plants' reservoirs play a key role in mitigating the peak outflow during flood events; reservoir management procedures, in fact, aim to maximise the energy generation so that, before rainy seasons, reservoir levels are lowered to increase the storage capacity of the incoming flood volume, regulating both the peak outflow during flood events and the turbinised outflow during drier season.

Within this framework the paper represents a clear example of water flood management capability of a dam reservoir studied combining the use of CFD analyses and classical hydraulics approaches. In the paper the overall design process, to determine the reservoir maximum level and the maximum outflow during design flood event, is shown, including the detailed analyses of the dam discharge facilities and the incoming flood routing calculations. CFD analyses were necessary to determine the behaviour of the overall discharge facilities composed by the morning glory spillways and the bottom and middle level outlets under contemporary working conditions.

1 Description of the hydraulic facilities

The dam studied is a clay core, 60 m high dam, in international context, with two morning glory spillways, a middle level tunnel spillway and a bottom tunnel spillway. The shaft spillways have a nominal discharge capacity of about 600 m³/s each. The middle level tunnel spillway has a nominal discharge capacity of about 360 m³/s and the bottom tunnel spillway has a nominal discharge capacity of about 200 m³/s. The bottom outlet is directly connected to the lower portion of one of the two morning glory spillways, and it discharges in the lower bend of one of the vertical spillways' shafts when both the morning glory and the bottom tunnel outlet are active with a scheme similar to the following:

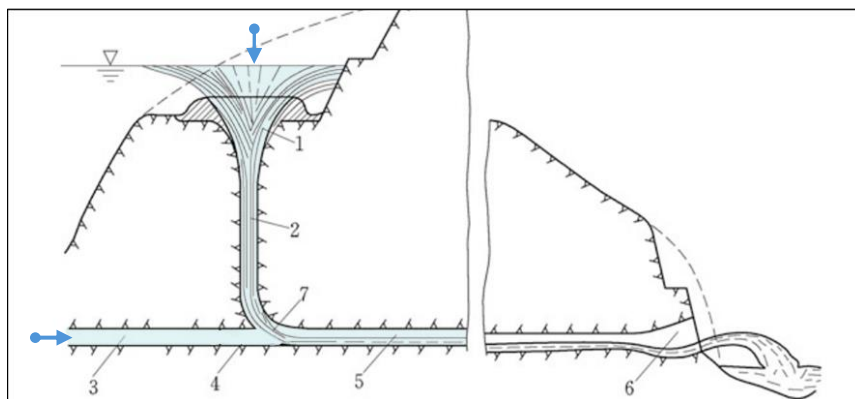


Figure 1 - Example of morning glory spillway with bottom outlet in the lower bend

The Figure 1, from ref. 1, shows a typical example of shaft or morning glory spillway with the presence of the bottom outlet in the lower bend of the shaft.

2 Methodology

The following steps have been carried out to study the flood lamination effect of the reservoir to determine the maximum reservoir level during the flood event:

- a preliminary hydraulic study was performed to determine the rating curves of the overall discharge

facilities not working at the same time. Theoretical rating curves for the two morning glory spillways, for the middle level outlet and for the bottom level outlet were determined applying the classical hydraulics formulation and verifying them with recorded measurements made available by the dam operator

- a CFD model of the overall discharge system including the morning glory spillways, the bottom tunnel and the middle level outlet was developed. The model was calibrated using the theoretical rating curves in their head ranges of validity and then exploited to define in detail the rating curve of the coupled facilities discharging the flow at the same time. The model allowed to study the behaviour of the morning glory spillway and the other outlets working simultaneously.
- the incoming flood hydrograph was available as an input, and it was not necessary to implement any further hydrological analysis
- the reservoir height – volume relationship has been determined starting from available historical data
- the routing model was implemented to solve the continuity equation of the reservoir knowing the incoming flow, the reservoir geometrical characteristics and the discharge facilities rating curves calculated through CFD model. The routing model allowed to define the outflows during the time of the hydraulic facilities and the variation during the time of the reservoir level starting from an initial water level condition at the beginning of incoming flood event
- through routing model, the reservoir water level variation in the time, the outflows variation in the time were determined.

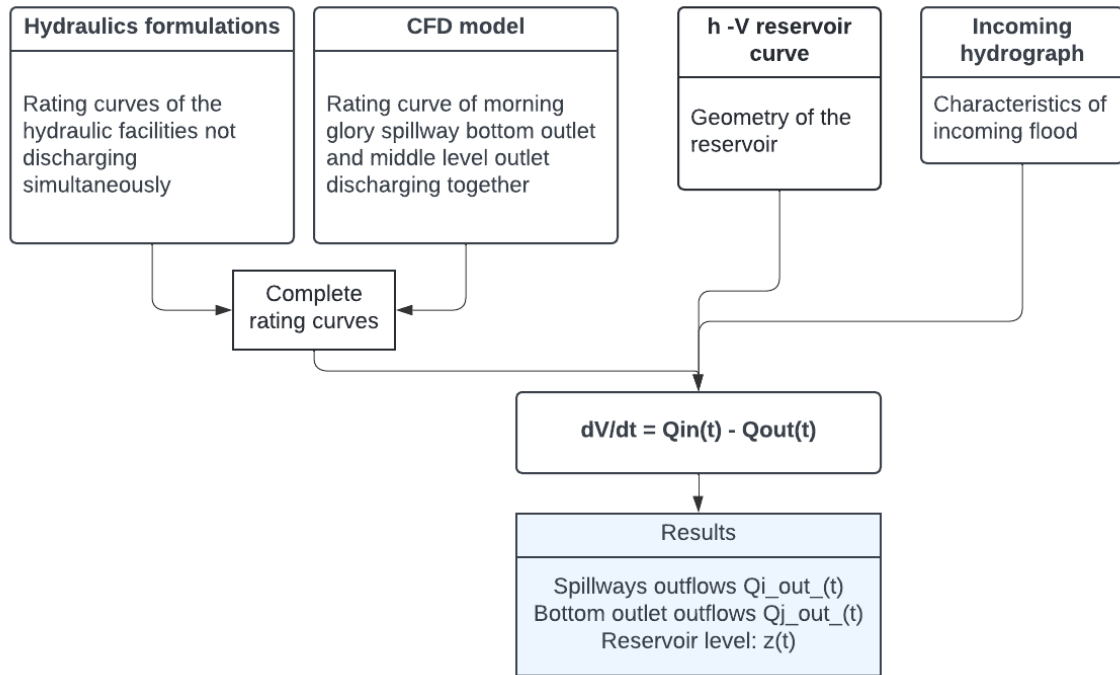


Figure 2 – Flow chart showing the steps adopted during the analysis

Figure 2 graphically illustrates the steps involved in analysing the reservoir's routing capacity.

3 Preliminary Hydraulic study of the discharge facilities

3.1 Shaft Spillways

The Figure 3 demonstrates that as the hydraulic head on the spillway crest increases, the hydraulic behaviour transitions progressively from crest control to tube control flow, and ultimately to full pipe flow. The different behaviours can be detailed as follows:

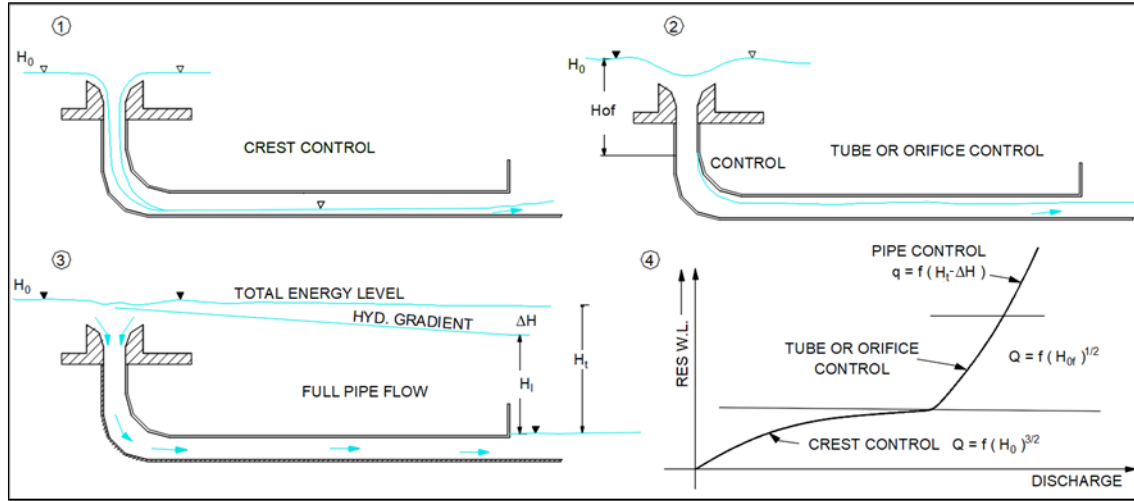


Figure 3 – Flow conditions and discharge characteristics ((USBR-1973)

The different behaviour, as shown in Figure 3 are:

- Crest control flow - the discharge is conditioned by the length of the crest and the head on the crest according to the relationship $Q = f(h^{3/2})$ where h is the hydraulic head on the spillway sill
- Tube or orifice control - the discharge is controlled by the vertical portion of the spillway and the relationship between the flow and the hydraulic head is given by $Q = f(h^{0.5})$, where h is the hydraulic net head on the centroid of the orifice. In this condition an annular flow jump, progressively rising to the surface, is created. For the specific case, based also on measurements interpolations, the following rating curve has been proposed:

$$Q = (147 * h^{1.59}) m^3/s$$

- Full pipe flow – when the annular jump rises the surface, the pressurized flow is fully developed in the tube; the discharged flow depends on the difference between the upstream and downstream water levels and from the head losses (distributed and local) in accordance with:

$$Z_{upstream} - Z_{downstream} = \frac{v_{downstream}^2}{2g} + \sum_{i=1}^j \lambda_i v_i^2 / 2g D_i * L_i + \sum_{i=1}^n k_i * v_i^2 / 2g$$

Where $v_{downstream}$ is the flow velocity at the downstream tunnel outlet, the second term refers to the distributed losses and the third term to the local losses; λ_i are the friction losses coefficients for the linear tube stretches with different diameter D_i , calculated in accordance with the Colebrook-White formulation, while the k_i coefficients refer to the local losses coefficients.

The Figure 7 shows both the rating curve for the crest behaviour and for the full pipe flow behaviour

3.2 Middle level outlet and bottom outlet

The middle level outlet is a conduit with a gate about 4 m high and 3 m wide while the bottom outlet is a conduit with a gate about 3 m high and 2 m wide. By calculating the head losses of the whole conduit and through comparison with available previous studies and measurements it was possible to determine the rating curves for the pressurized behaviour of the outlets for different gates openings when not discharging at the same time with the morning glory spillways. The curves of these two discharge outlets are not shown but their contribution to the total outflow is described in detail in the results of the overall CFD model presented in the paragraph 4.

3.3 Limits of the classical hydraulic analysis

Although, with the classical hydraulics formulations, it was possible to determine the rating curves of all the hydraulic facilities, when not discharging at the same time, it was not possible to determine how the morning glory spillway and the bottom outlet spillway influence each other when both are active taking into consideration that they have in common the horizontal tunnel below the shaft bend. For this reason, it was decided to implement a CFD model using the calculated rating curves for the calibration of the model.

4 CFD Modelling

The study involved a preliminary CFD model calibration through the comparison of the model outcomes with the results of the theoretical approaches in their validity ranges; then the model has been exploited out of the validity range of the classical hydraulic formulations. The study gave the opportunity to describe in detail the spillways

behaviour and the complete discharge capacity curve of the overall release facilities when simultaneously working; furthermore, it was also possible to consider the effects of the reservoir shores located near the spillways and the anti-vortex walls radially disposed on the spillway crest. In addition, different simulations were implemented to analyse the transition between the free and the submerged flow condition for the shafts' spillways completely describing the rating curve.

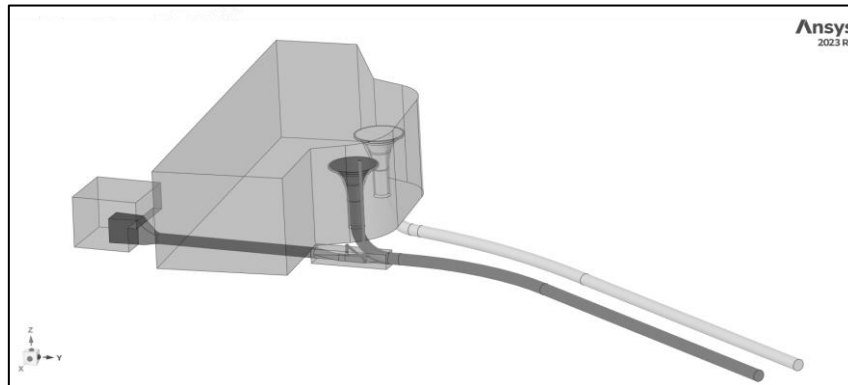


Figure 4 – Solid models of the hydraulic facilities

4.1 Geometry and mesh

The 3D models of the two morning spillways and the bottom outlet were imported in the hydraulic calculation model as shown in Figure 4 and Figure 5 identifying the different volumes corresponding to the hydraulic structures and to the reservoir. In these figures it is possible to see the bottom outlet with the discharge channel in common with the morning glory spillway, the intake structure of the bottom outlet, and a portion of the reservoir upstream of the spillways. The volumes are automatically meshed by the calculation program. The Figure 6 shows the difference of the calculation domain before and after application of the adaptive mesh function.

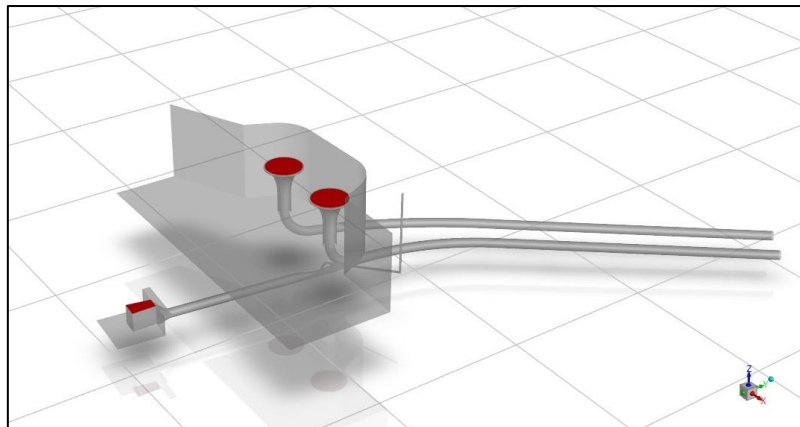


Figure 5 – Solid models of the hydraulic facilities

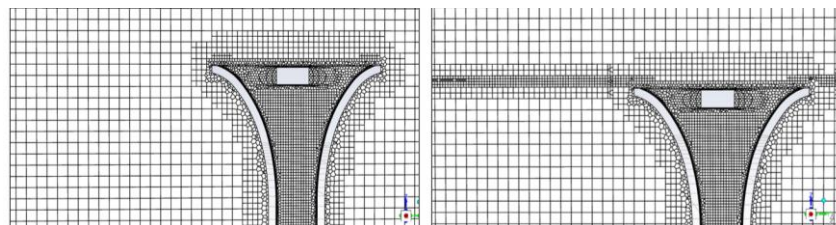


Figure 6 – adaptive mesh function

4.2 Boundary conditions

The boundary conditions have been applied both in the upstream reservoir and in the outlet canal, collecting the discharge of the hydraulic facilities. Upstream Boundary conditions have been applied to the lake surface, and to the inlet sections of the hydraulic facilities. For the bottom surfaces of the lake a “pressure condition” corresponding to the hydrostatic pressure value has been applied. For the wall a “standard roughness model” has been applied considering an absolute roughness coefficient ε/D equivalent to a value of Gauckler-Strickler

roughness coefficient equal to $70 \text{ m}^{1/3}/\text{s}$ as a first approximation then verified during the calibration of the model. The following table summarizes the boundary conditions adopted for the models:

Location	Type	Property
Surface of the lake	Pressure inlet	Open Channel yes
		Total Height [m a.s.l.] Water level in the reservoir
		Velocity magnitude [m/s] 0
		Bottom level [m a.s.l.] Bottom elevation
Bottom reservoir surface	Pressure inlet	Open Channel No
		Gauge total pressure [Pa] Pressure at bottom elevation
		Water volume fraction [-] 1
Bottom reservoir surface	Symmetry	Symmetry condition for the velocity vector, no velocity vector crossing the surface
Outlet surfaces	Pressure outlet	Open Channel No
		Gauge pressure [Pa] Atmospheric pressure
Downstream basin	Pressure outlet	Open Channel No
		Gauge total pressure [Pa] 0
		Water volume fraction [-] 0
Atmosphere above upstream and bottom reservoir	Pressure inlet	Open Channel No
		Gauge total pressure [Pa] 0
		Water volume fraction [-] 0
Walls of the hydraulic structure	Wall	Roughness model Standard

Table 1 – Summary of the boundary conditions applied to the calculation model

4.3 Model calibration

A separate model of the morning glory spillways, without the bottom outlet, has been adopted for the calibration of the model to have the possibility to compare the model results with the theoretical rating curve of the spillway and refine where necessary the mesh sizing and the roughness coefficient.

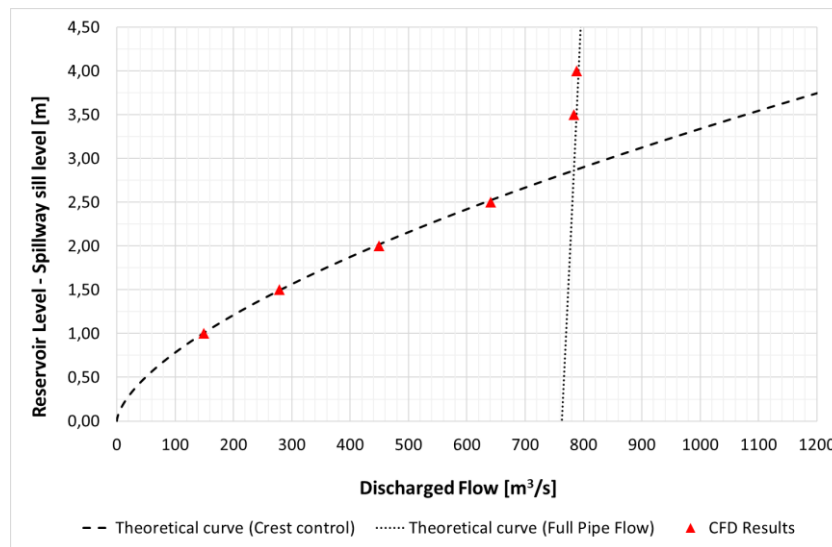


Figure 7 – Comparison between model results and theoretical formulas

Different simulations, with different reservoir water level have been implemented, and for each simulation the flow discharged by all the hydraulic facilities has been recorded. The couple of values reservoir level and discharged flow was compared with the theoretical results as shown in Figure 7. It is possible to see how the results in terms of discharged flow for different water level in the reservoir match with the theoretical curves for crest control and full pressurized flow in the spillway as derived according to par. 3. With an error less than 1% on the water flow discharged, the parameters of the CFD model used for the calibration process have been adopted for the main calculation model.

4.4 Analysis of the results

Different simulations, each one with a different water level in the reservoir, have been carried out and for each of them the discharged flow of the hydraulic facilities has been recorded obtaining the complete total rating curve of the overall system and the single contributions of all components. In this way it was possible to verify the influence of the simultaneous discharge of the morning glory spillway on the bottom outlet and the difference in the

behaviour between the morning glory coupled with the bottom outlet and the not – coupled one. An example of the results obtained is shown in Figure 8 where it is possible to compare the rating curve of the morning glory (flow through inlet section) with and without the presence of the bottom outlet. The points in the graph correspond to the different simulations implemented.

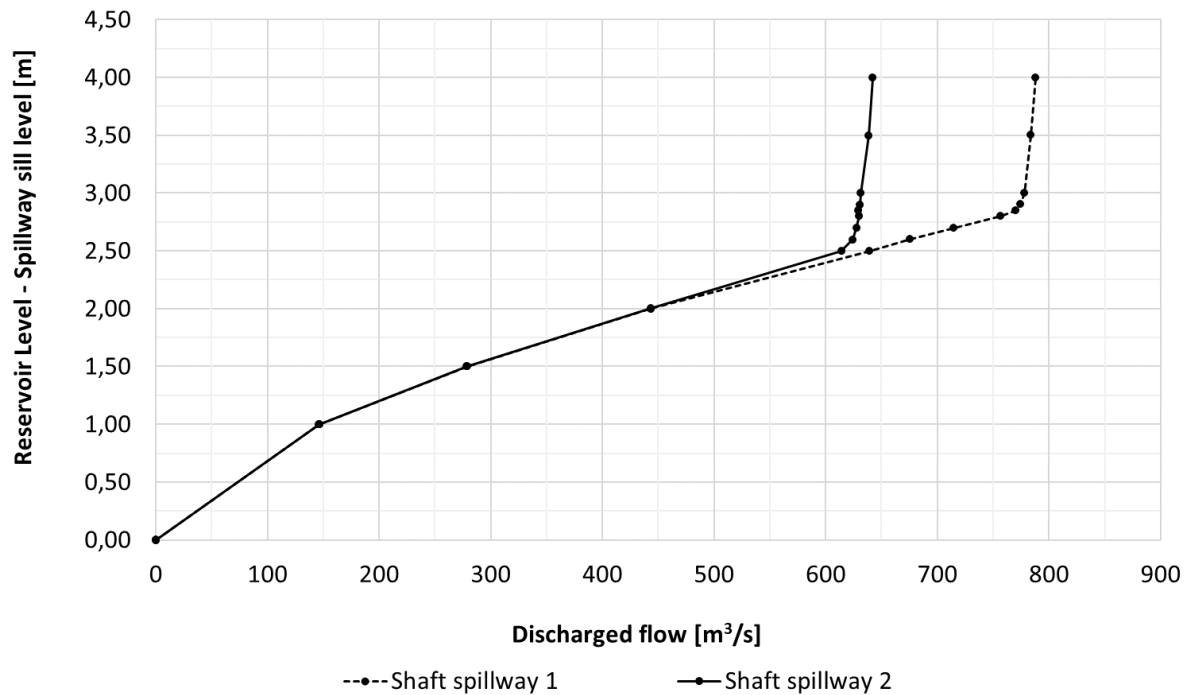


Figure 8 – rating curves of the shaft spillways only with and without the coupling with the bottom outlet

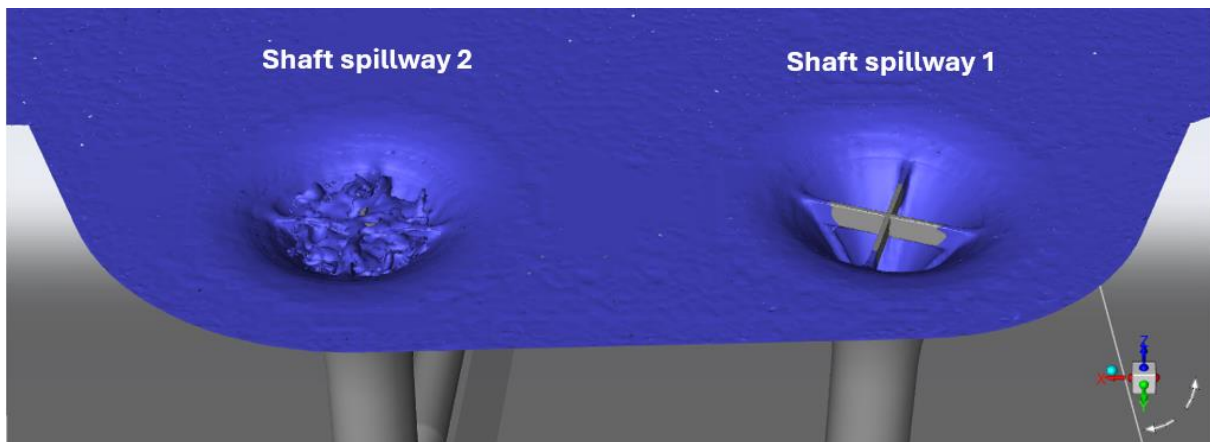


Figure 9 – Image of the two morning glory spillways. The one on the left, connected to the bottom outlet, reaches the saturated condition before than the other one.

It is possible to note that up to reservoir elevation corresponding to 2 m water height on the spillway sill the behaviour of the two spillways is the same, meaning that for low heads on the shaft crest the presence of the bottom outlet does not have a significance influence on the shaft spillway, then for water elevation higher than 2 m, the presence of the bottom outlet, discharging in the lower part of the morning glory spillway, reduces the discharge capacity of the latter which reaches the saturation point at elevation about 2,60 m of water height on the shaft sill, against 3 m of the spillway not coupled with the bottom outlet. Such difference due to the reciprocal influence of two outlets coupled was not easily appreciable with classical hydraulics calculations while it is clearly highlighted by the CFD modelling. The Figure 10 shows the comparison between the cumulated rating curve of the shaft spillway + bottom outlet and the rating curve of the sole shaft spillway without bottom outlet. It is possible to note that the total flow discharged by the coupled discharge facilities is higher, even in saturated condition, than the discharge of the single morning glory. If, on one side the presence of the bottom outlet induces earlier saturation of the morning glory spillway, on the other side the two outlets discharge together a flow higher than the flow discharged by the other morning glory not connected to a bottom spillway. Another interesting result presented in Figure 10 is the slope variation of the cumulated rating curve of the shaft spillway and the bottom outlet together

working for 2 m water height above the shaft sills; the contribution of the bottom outlet increases the total discharge up to saturation condition reached, for the coupled system, at a water level lower than the shaft spillway without connection to the bottom outlet. The presence of the bottom outlet accelerates the shaft spillway saturation process because of the additional flow discharged in the lower part of the spillway and not present for the other shaft spillway. The total rating curve due to the contribution of the two shaft spillways, the bottom outlet working at the same time is obtained summing, at same water level, the single contributions as obtained by CFD analyses.

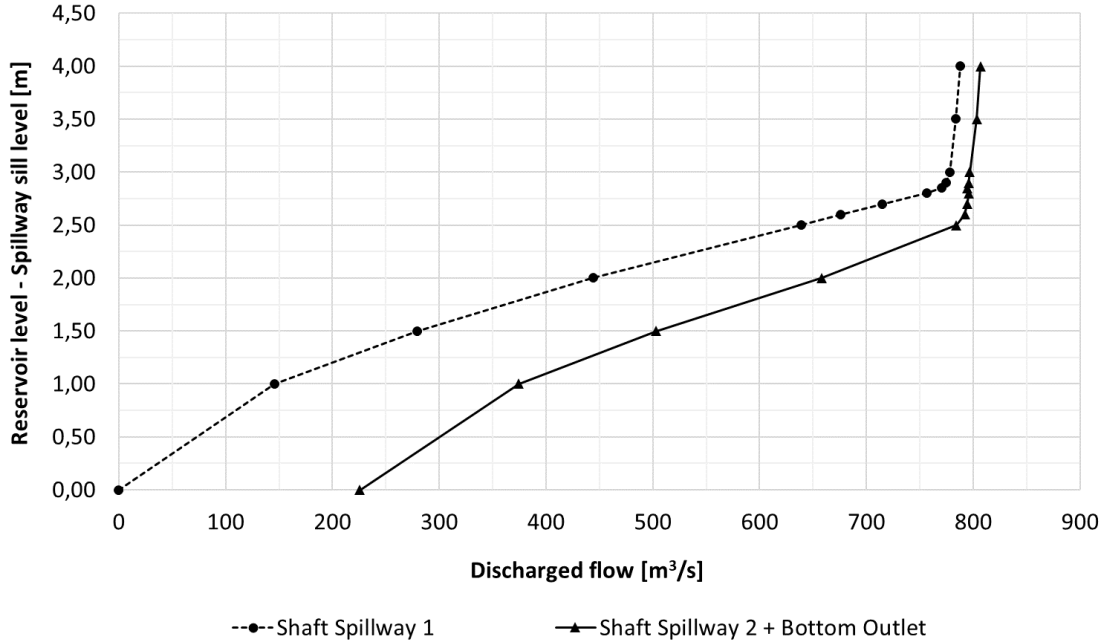
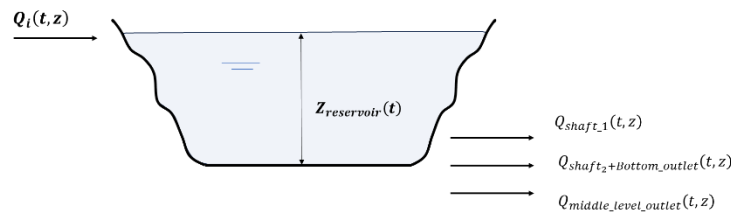


Figure 10 – rating curves of the shaft spillway 2 coupled with the bottom outlet (both contributions) and rating curve of the shaft spillway 1 without the bottom outlet

5 Study of the reservoir routing capability

Starting from the complete rating curves determined through the CFD models, the reservoir height – volume curve, and the incoming flood hydrograph made available to us as an input data it is possible to study the routing capability of the reservoir by solving the continuity equation of the reservoir:



$$\frac{dV}{dt} = Q_i(t, z) - Q_{shaft1}(t, z) - Q_{shaft2+B.O.}(t, z) - Q_{M.L.O.}(t, z)$$

Figure 11 – continuity equation for the reservoir routing capability study

Where $Q_i(t, z)$ corresponds to the known inflow hydrograph, and $Q_{out_i}(t, z)$ – the sum of the other flow rates, is obtained from the known rating curves of the discharge facilities starting from the reservoir level at time t . The continuity equation states that the variation of the reservoir volume between t and $t + dt$ is equal to the difference between the inflows and the outflows at time t multiplied for the time interval dt . A numerical explicit solution can be obtained for the reservoir equation by substituting the derivative of the reservoir volume function with the incremental ratio and calculating the volume at $t + dt$ as the volume at time t plus the average of the difference of the known inflows and the known outflows at time t and $t-dt$:

$$V_{(t+dt)} = V_{(t)} + \Delta V_{(t)}^{calc} = V_{(t)} + \Delta t * \left(\frac{Q_{(t-dt)}^{in} - Q_{(t-dt)}^{out}}{2} + \frac{Q_{(t)}^{in} - Q_{(t)}^{out}}{2} \right)$$

In the equation the value to be determined is the volume of the reservoir at time $t + dt$. With such kind of discretization, it is possible to obtain results like the one presented in Figure 12:

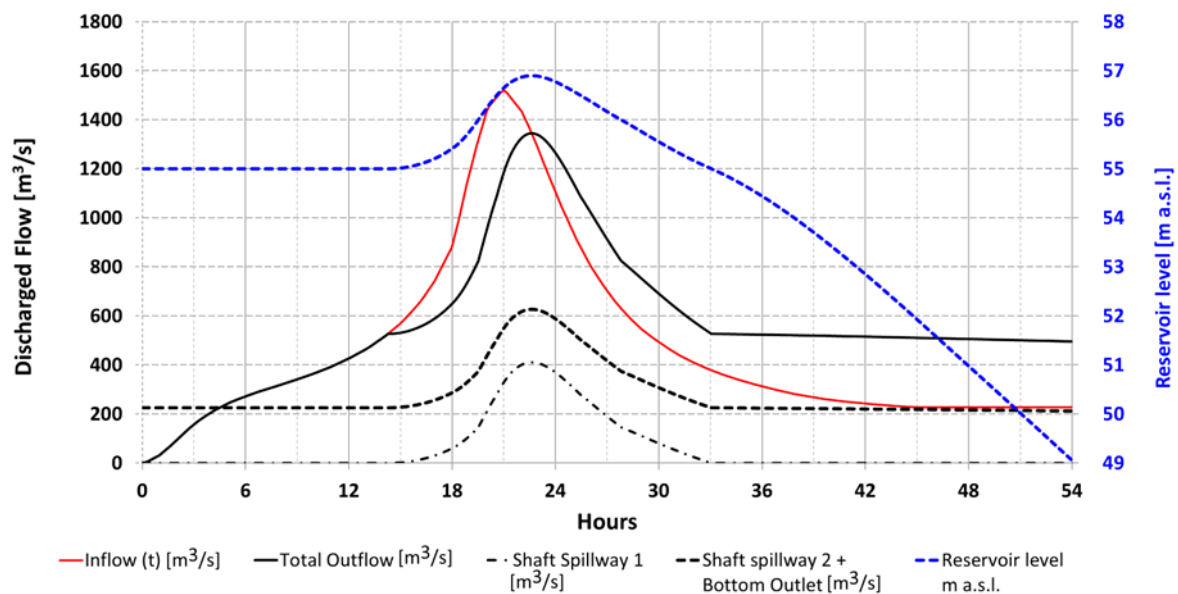


Figure 12 – Typical results of reservoir routing analysis

The graph shows, as a function of the time t , the water level in the reservoir, the total inflow, the total outflow and the outflows of all hydraulic facilities. From the graph it is possible to see that the level in the reservoir reaches the highest value when the total outflows equal the total inflows. Starting from this moment, being the total outflow higher than the total inflow, the water level in the reservoir decreases. With the same model it is possible to study all the possible scenarios by changing the opening sequence of the hydraulic spillways and find the optimum operation mode to minimize the maximum level reached in the reservoir.

6 Conclusions

The paper presents an application of CFD modelling coupled with a routing model to analyse the routing capability of a reservoir exploited for hydropower energy generation. CFD modelling has been effective to determine the rating curve of a complex spillway system constituted by a morning glory spillway connected, at the lower bend, with the bottom outlet gallery. The CFD model has been firstly calibrated through the comparison of a simplified sub model results with the rating curve obtained by the classical hydraulic formulations: this process allowed the suitable mesh refinement and the roughness coefficient optimization. The main model, once modified in accordance with the calibration process, has been exploited out of the validity range of the classical formulations to study the behaviour of the shaft spillway simultaneously discharging with the bottom outlet. Through a series of simulations performed for different reservoir levels the rating curves of the hydraulic facilities have been determined and then used in a routing model to verify the routing capability of the reservoir in managing the income volume of a design flood hydrograph. Through the routing model it was possible to determine the variation in time of the water level in the reservoir, of the incoming flow and the outgoing flow discharged by the shaft spillways, the bottom level outlet and the middle level outlet. The CFD analyses were effective in assessing the hydraulic characteristics of the complex facilities and ensuring compliance with international standards regarding dam crest freeboard and shaft spillway saturation levels.

References

1. R. M. Kathuria, Hydraulics of spillways and energy dissipators, New York, 2005
2. Bureau of reclamation, Design of small dams, A water resources technical publication, 1987
3. Bureau of reclamation, Design of small canal structures, A water resources technical publication, 1978
4. American society of civil engineering, Morning glory – Shaft spillways, A symposium, 1956
5. U.S. Army Corps of Engineers, Hydraulic Design of spillways, Washington, 1992

The Authors

Simone Citterio: hydropower engineer, involved in hydropower since 2000 from feasibility studies to detailed design phase, He is currently engineering division manager in FROSIO NEXT.

Luigi Papetti: hydraulic and chemical engineer, involved in the design and supervision of hydro-power plants since 1990. He is currently Chief Executive Officer of FROSIO NEXT.