MODELING OF FIERZA DAM SPILLWAY OPERATION AND DOWNSTREAM EFFECTS ON RIVER BED AND SLOPES

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Abstract. This paper regards the case study of the two spillway tunnels of the 166.5 m high Fierza hydropower dam, in the Drini River, Albania, operated by KESH sh.a. since 1978. The analysis focused on both spillway tunnels and their jet flows impact on the river bed, bank slopes and on downstream flow conditions.

The assessment of spillways operation under different discharge combination was performed by LNEC, Portugal. A 1:80 scale physical model and both 1D and 3D numerical models were applied. A set of recommendations for remedial measures to prevent observed slope instabilities and erosions was produced, as well as an evaluation the flow interference with the existing road bridge located approximately 500 m downstream of the dam toe.

This paper summarizes the performed tests/simulations and respective results. Furthermore, it is intended to evidence the advantages of a combined approach using traditional hydraulic physical models together with novel CFD numerical models, which are growing in hydraulic structures applications.

INTRODUCTION

Fierza dam is the highest dam in Albania (Figure 1), located in its northern region as the upstream most dam of the Drini River cascade, which the main purpose is hydropower production. It creates a 2.7 billion m^3 artificial lake and inundates an area of 72 km^2. Its construction began in 1971, having entered in full operation in 1978, by then was the second tallest dam in the Europe of its type. The Drini River cascade, involving 3 hydroelectric power plants (HEPP), is administrated by KESH (Albanian Power Corporation), the largest hydropower producer in the Balkan region [1].

Considering the installed power, location and lake volume, Fierza HEPP plays a key role in the exploitation, regulation and operation of the Drini River cascade. The height and type of the dams on the Drini River, the created reservoirs, involved power and dynamic management of power plants make this cascade unique in Europe.

Fierza dam is a 166.5 m high and 380 m long rockfill dam with a clay core. Its width ranges from 576 m at the base to 13 m at the crest. The power plant located at the dam toe on the left bank has 4 turbines and a maximum flow discharge of 500 m^3/s which is conveyed from the reservoir through tunnels.

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Figure 1: Fierza Dam. View of downstream zone from the crest

Fierza dam is classified as a first class dam in term of risk, being designed for a return period of 1,000 years (6,100 m^3/s) and for a verification flood of 10,000 years (6,400 m^3/s). The excess flood flows are discharged downstream by means of a two spillway tunnels designed for a maximum outflow discharge of 2,670 m^3/s.

BACKGROUND REGARDING SPILLWAY OPERATION

Dam spillway is placed on the right side of the dam, being composed of two tunnels labeled as No. 3 and No. 4 (tunnels No. 1 and No. 2 were those used for river diversion for dam construction). The tunnels are located at different levels: the inlet and outlet elevations of spillway No. 3 being lower than the respective inlet and outlet of spillway No. 4.

Capacities of spillways No. 3 and No. 4 are respectively 1780 m^3/s and 890 m^3/s. The inlet of spillway No. 3 is in a reinforced concrete tower in the reservoir accessible from the dam crest through a metal suspension bridge, while the intake of spillway No. 4 is accessible from a stairway at the dam right abutment. Each spillway discharge is controlled by one radial gate, 7x7 m^2 on spillway No. 3 (Elevation 222 m) and 6x8 m^2 on No. 4 (Elevation 268.4 m). Maintenance gates are also provided in each intake.

Presently, discharges are made solely through spillway No. 4 due to several problems that have occurred with the gate of spillway No. 3 thirty years ago. Then, also massive concrete protection elements placed in the river bed where the discharged flow from both spillways used to fall were destroyed and displaced towards a bridge on the Drini River located approximately 500 m downstream the spillways exit structures (ski jumps).

In addition, the spillway discharges since dam entered into operation partially damaged the slopes in the opposite left bank down to the area of the already mentioned bridge (Figure 2).

After operation with spillway No. 4 in December 2010, an extension of the bridge was introduced by adding two additional spans near the right bank, as well as a protection of slopes on the bridge left abutment vicinity (Figure 3). In the meantime, the area between existing bridge and new spans was again damaged during the operation of spillway No. 4 in 2016 (Figure 4). Another problem to be considered derives from the fact that, during operation with spillway No. 4, the bridge deck interferes with the flow, creating problems at power house facility of Fierza HEPP where the water elevation is limited to 176 m.

Figure 2: Jets' alignments (cyan) on satellite images. 2003 with three span bridge (left); 2013 with eroded left bank and two additional bridge spans

Figure 3: Left bank protected area (left) and two new bridge spans (right)

Figure 4: View of Discharges of 600 m^3/s from Spillway No. 4, November 2016 and Waves on the left bank protection (left) and associated damages associated to Spillway No. 4 in operation at 50%

Taking into account the observed changes in the bridge area configuration, its current conditions, the problems encountered since dam entered into service, the protection of the area on the left side of the lower section, including as well the power house diffusers and building, it became imperative the definition of protective measures throughout the downstream area of the dam. These should consider spillway operation using both tunnels. In order to ensure the effectiveness of measures, it was considered the development of numerical and physical models of spillways No. 3 and No. 4, as well as the plunge pool zone, bridge and downstream river stretch influencing the flow conditions. Models' conclusions and respective recommendations support remedial measures, namely regarding the performance of protective works and adjustments to the downstream area.

Prior to modeling it was made a thorough characterization of the current situation concerning river bed scour, bank's erosions, protective concrete blocks used during the construction works displacement and an analysis of previous studies on these topics [2].

HYDRAULIC MODEL STUDIES - METHODOLOGY

Numerical and physical modeling were executed in accordance with the following phases:

- geometrical definition of natural boundaries (bathymetry) and hydraulic structures;
- numerical 1D model to simulate flow between dam and Valbona River confluence;
- numerical 3D model to simulate complex spillway jet flows and plunge pool area;
- physical model for predefined scenarios and testing remedial measures, involving interaction between flow and erodible boundaries.

NUMERICAL MODELS

One dimensional numerical model

A HEC-RAS (US Corps of Engineers) one dimensional numerical model covering a river stretch of 1,875 m length between the power house of Fierza Dam and the confluence with the Valbona River was used (Figure 5). Upstream boundary condition scenarios considered multiple combination of the discharge from the spillway tunnels and the power house. Downstream boundary conditions consisted of pre-defined water levels associated to the Koman reservoir water elevation. Simulations were performed with and without the bridge.

Figure 5: Plan view of HEC-RAS 1D numerical model (left) and 3D view of the bridge zone (right)

Regarding the downstream water level at Valbona River Confluence, it was concluded that:

- for elevation175 m downstream - even for lowest discharge of 445 m^3/s (spillway No. 4 at 50%) the water level reaches the bridge beam;
- for elevation 171 m downstream – for a discharge of 1,835m^3/s (spillway No.3 and No. 4 at 50% and power house at 100%) the bridge beam is reached by the water surface;
- the bridge deck is affected by the flow for discharges above 2,000 m^3/s regardless of the elevation imposed at downstream boundary.

Results evidenced that the river stretch between the bridge and Valbona River confluence includes zones with very limited discharge capacity. Furthermore, the influence of the bridge in the water surface profile is negligible. The opposite doesn't apply, *i.e.*, the bridge is quite affected by the spillway and power house discharges, even at low discharge values.

Numerical 3D Model

The numerical 3D model was set up to simulate the jet flows produced by the operation of spillways No. 3 and No. 4 and assess the downstream impacts in the plunge pool and over a Drini River stretch that includes the road bridge. CFD software OpenFOAM with IHFoam solver was adopted ([3], [4] and [5]), allowing to model turbulent free surface, water-air flows, incompressible and immiscible two-phase fluid.

Unsteady Reynolds Average Navier-Stokes (URANS) was considered for turbulence modeling and Volume-of-Fluid (VOF) method was considered to track down the free surface position.

The following boundary conditions were assumed: closed plunge pool upstream reach; closed boundary regarding water and air flux; plunge pool downstream boundary, either closed to maximize the return currents near the banks in the plunge pool zone), or open to maximize flow velocity in road bridge section; in-out open top domain boundaries; ski jump entering flow conditions as in Table 3 from external hydraulic calculations.

		tunnel exit - ks = 70 m1/3 /s			
Relative opening ar=a/a0 (-)	Q/Qmax (%)	HTExit (m)	hTExit (perp to floor) (m)	UTExit (m/s)	Q (m3/s)
1.00	**100%**	250.37	**7.96**	**31.85**	1,774.43
0.84	**75%**	248.50	**6.06**	**31.38**	1,330.82
0.65	**50%**	247.95	**3.98**	**31.84**	887.22

		tunnel exit - ks = 70 m1/3 /s			
Relative opening ar=a/a0 (-)	Q/Qmax (%)	HTExit (m)	hTExit (perp to floor) (m)	UTExit (m/s)	Q (m3/s)
1.00	**100%**	266.68	**5.15**	**28.99**	895.00
0.80	**75%**	262.93	**3.98**	**28.14**	672.87
0.60	**50%**	244.19	**2.80**	**26.58**	446.92
0.32	**25%**	245.04	**1.63**	**22.77**	223.30

Table 3: Flow main characteristics at ski jumps approach. spillway No. 3 (left) and spillway No. 4 (right)

Mesh refinement tests were performed in order to achieve a reasonable compromise between calculation effort/time and results quality/resolution. A mesh with cells size ranging from 0.5 to 2 m was adopted, the most refined zones being in the ski-jump (geometries in Figure 6), along free jet trajectory and at the plunge pool free surface interface. A perspective view of the numerical domain is shown in Figure 7.

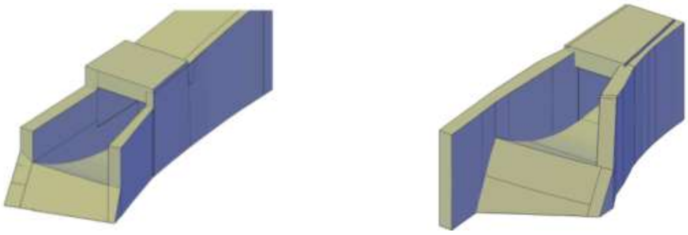

Figure 6: Spillway tunnels ski jump geometries. spillway No. 4 (left) and spillway No. 3 (right)

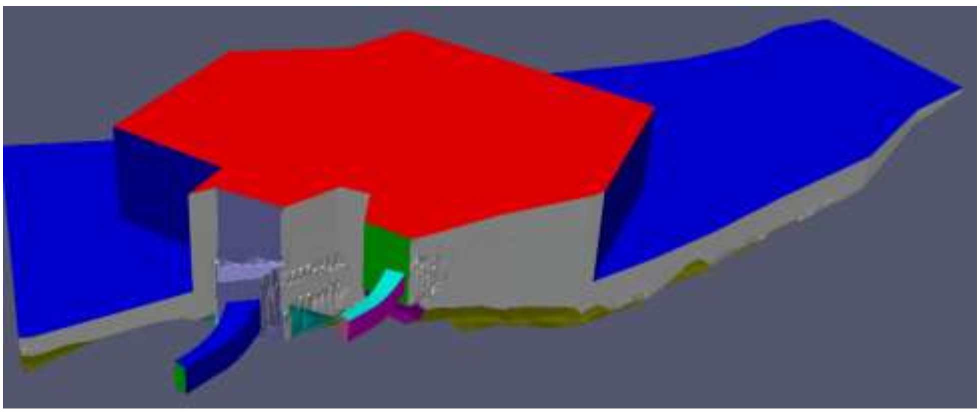

Figure 7: Perspective of the numerical domain from the left bank looking downstream

Simulation times ranged from 120 to 1600 seconds. Longer simulations were developed to analyze convergence conditions, namely for downstream open boundary scenarios. The numerical modeling results regarding jet throwing distance, angle of impact on plunge pool and impact velocities evidenced a reasonable agreement between prototype observation, analytical calculations and numerical simulation. Ten different operation scenarios were considered, as depicted in Table 4.

Some representative 3D graphs of flow simulations for a relevant operation scenario, i.e., joint operation of the spillways No. 3 and No. 4 for maximum capacity is presented in Figure 8, in order to evidence obtained flow patterns and velocity fields.

Scenario	Spillway	Discharge (m^3/s)	% of full capacity	Downstream boundary condition
T4_25	No. 4	224	25%	Closed
T4_50	No. 4	447	50%	Closed
T4_100	No. 4	895	100%	Closed
T4_100_open	No. 4	895	100%	Open
T3_50	No. 3	887	50%	Closed
T3_50_open	No. 3	887	50%	Open
T3_100	No. 3	1774	100%	Closed
T3_100_open	No. 3	1774	100%	Open
T3_T4_100	No. 3 + No. 4	2669	100% + 100%	Closed
T3_T4_100_open	No. 3 + No. 4	2669	100% + 100%	Open

Table 4: Simulated operation scenarios

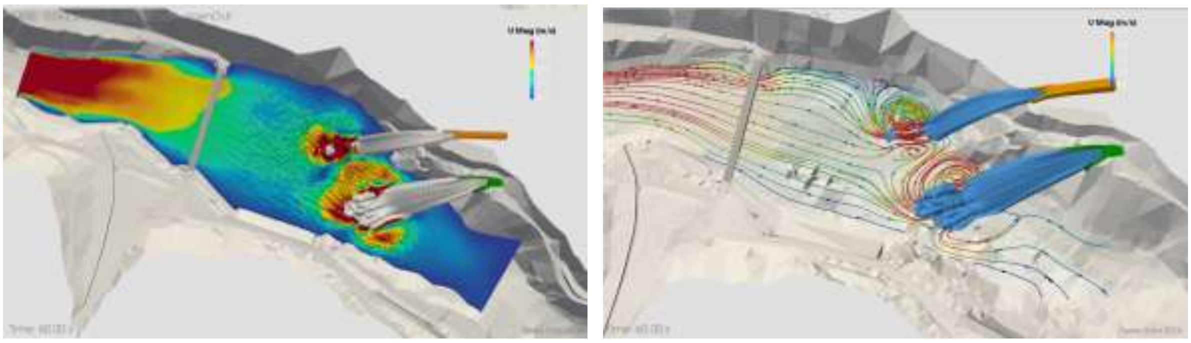

Figure 8: Jet and plunge pool velocity magnitudes and velocity streamlines in plunge pool - both spillways operating at full capacity. Open downstream boundary.

For each simulation scenario, velocity and free surface elevation were recorded at 14 gauging predefined locations. The generated meshes were not sufficiently refined to reproduce the plunge pool free surface waves. Therefore, analysis of the water surface agitation, turbulence, circulation currents and river bed/bank slope stability were addressed in the physical model.

Simulation of only spillway No. 4 operation confirmed that the left bank damaged areas near the bridge abutment were produced by the spillway operation, as the flow pattern in the plunge pool evidence that the streamlines impinge directly in this area.

Upon adjustments of numerical model wall parameters, based on the physical model results, numerical model simulations produced return currents with flow velocity reaching 5 m/s near the most sensitive zone of the left bank slope under spillway No. 3 operation, this result being in line with the physical model results (presented in section 5).

Open downstream boundary simulations evidenced the strong flow constriction imposed by the river cross section at bridge location, namely for operation of spillway No. 4 at full capacity, for spillway No. 3 above 50 % capacity and for joint operation of No. 3 and No. 4 spillways. This situation most likely derives from the fact that the river cross section in locations downstream of the bridge are too narrow and/or too shallow, strongly influencing the river flow capacity. The 1D numerical model already evidenced this limitation and it was also confirmed in the physical model tests, as described in section 5.

PHYSICAL MODEL

General description

The physical model was built at a geometrical scale 1:80 and was explored in accordance with Froude similitude, as depicted in Figure 9. Tests were performed considering 9 hours of steady operation (prototype value). Local velocities, free surface agitation, water levels at reference sections, scour in plunge pool and erosions along the left bank slope toe were measured or observed at predefined locations. The physical model operation scenarios that were tested are presented Table 5.

Figure 9: General views of the physical model looking upstream (left) and downstream (right)

	Test ID	Tunnel No.3 % Qmax	Tunnel No.4 %Qmax	Power House %Qmax	Comment
Set 1	1	0%	50%	0%	Operation **not imposing** any downstream control in the model (model flap gate fully open)
	16	50%	0%	0%	
	10	0%	75%	0%	
	3	0%	100%	0%	**Sufficient bridge freeboard**
Set 2	2	0%	50%	0%	Operation **imposing** a control downstream that leads to the situation of **no bridge freeboard**
	17	50%	0%	0%	
	11	0%	75%	0%	
	4	0%	100%	0%	
Set 3	13	75%	0%	0%	Operation **not imposing** any downstream control in the model
	5	50%	50%	0%	**No bridge freeboard**
Set 4	18	50%	0%	0%	
	12	0%	75%	0%	
	15	75%	0%	0%	Operation imposing elevation **175 m** near the power house
	6	50%	50%	0%	
	7	0%	100%	0%	**No bridge freeboard**
	8	50%	50%	100%	
	9	100%	100%	0%	
Set 5	E	50%	50%	100%	**Left bank with riprap protection**
	H	100%	100%	0%	Bridge deck removed
	I	100%	100%	100%	Operation imposing elevation **175 m** near the power house
	Z	0%	50%	100%	Operation conditions close to the ones of 14 November 2016

Table 5: Physical model tested scenarios

The fifth set of tests, labeled with capital letters, refers to the analysis of the protection of left bank slope using different zones of specific riprap protection along left bank slope toe. An additional isolated test was also performed in which the imposed hydraulic operation conditions were similar to those that occurred in 14 November 2016, that caused significant damages on the bank protection near the bridge and on the bridge itself (Figure 4).

Simulation of existing situation

Simulation tests of existing situation (sets 1, 2 and 3 of tests), were performed to assess the flow erosion capacity, associated velocities and the wave heights along the left bank. The sand materializing the slopes was placed in the model as depicted in Figure 9. In Figure 10 general views of the model for two operation scenarios (Tests 4 and 9) are presented, as well as corresponding situations from 3D numerical model for comparison.

Graphical representation of the observed erosion for the most significant scenarios based on photogrammetric survey is presented in Figure 11. The erosion along the left bank in the transition from the unprotected to the protected slope (red arrow in figures) is particularly severe when spillway No. 3 operates, whilst with spillway No. 4 it is not as severe, although some material is still removed from the slope toe.

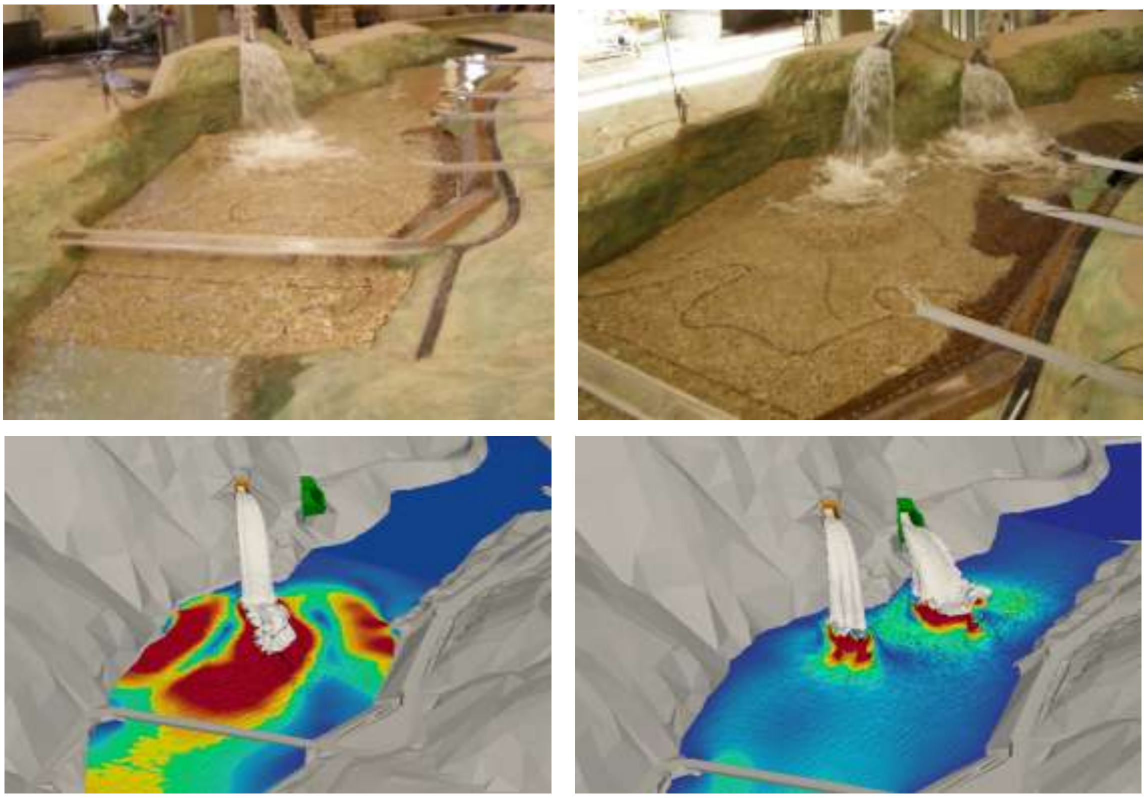

Figure 10: General view of physical and 3D numerical model for Test 4 (left) and Test 9 (right)

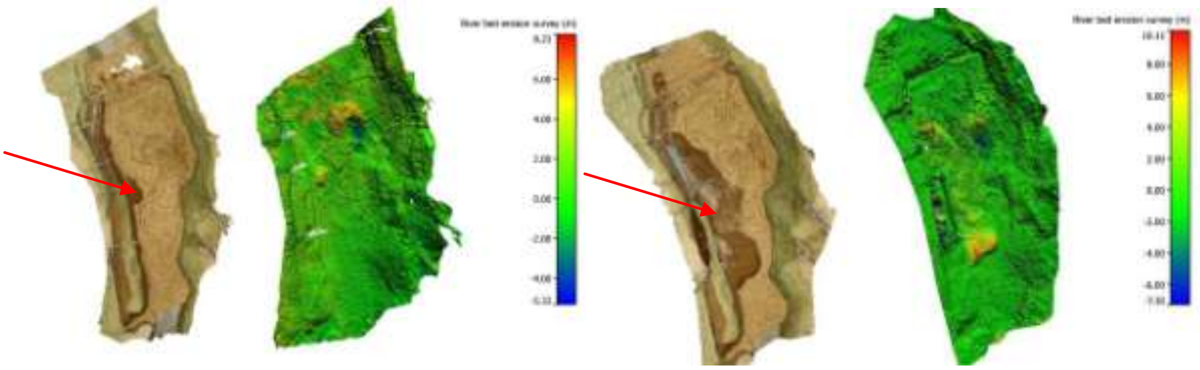

Figure 11: Erosion results for existing situation. Test 4, spillway No.4 at full capacity (left) and Test 9, spillway No. 3 and No. 4 at full capacity (right)

The sets 2 and 3 of tests allowed to assess the hydraulic operation conditions that lead no freeboard below the bridge deck, assuming Koman reservoir is very low. It became evident that for discharges over approximately 1 300 m^3/s, no freeboard under the bridge is observed anymore, confirming the conclusions achieved with the numerical 1D model.

Measurements of velocity were performed at 9 points next to the left bank slope, where the erosion where found. A maximum velocity of 6.1 m/s located in front of spillway No. 3 occurs in test 6. As mentioned in section 3.3, the numerical 3D model, after some calibration adjustments, lead to a maximum flow velocity of 5 m/s in the same zone.

Five free-surface probes, displaced along the left bank slope measured free surface agitation intensity produced by spillway tunnels operation. The location of most severe agitation occurs for Test 9 (spillway at full capacity) in front of spillway No. 3.

Simulation of a riprap protection of left bank slope

Based on design criteria for establishing riprap block size based either on flow velocity [6] or on wave characteristics [7], a flexible riprap protection was tested to prevent the erosion of left bank slope toe. The criteria pointed out for riprap blocks with 1.45 m to 1.60 m of mean diameter in the most severely affected zone of the bank slope toe. For the adjacent zones downstream, it pointed out to mean diameters of 0.66 to 1.00 m and for the left bank zone immediately upstream the bridge to a median diameter of 0.40 m. A riprap protection was constructed in the model respecting these mean sizes and considering a slope of 26° (Figure 12). As it can be seen in this figure, despite the severe hydrodynamic actions near the left bank slope, notably due to spillway No. 3 operation, only minor erosions were observed in the larger riprap (red).

Figure 12: Left bank slope protection zones considering heavy riprap (red), intermediate riprap (yellow) light riprap (blue). Situation before testing (left) and after Test H (Right)

CONCLUSIONS

The results from the numerical and physical models allowed reaching the objectives of the study, despite the unavoidable incertitude and limitations associated to each of them.

The numerical 1D model was quite useful to assess the interference between the existing bridge and the flow from spillway/power house operation, namely the relevance of the rating constraints imposed by the river stretch between the bridge section and the confluence with the Valbona tributary.

The numerical 3D model anticipated that the river capacity downstream of the bridge posed significant constraints to the plunge pool water elevation. After some parametric adjustments based on physical model results, the model became capable of reproducing the strong circulation currents potentially affecting the banks, mainly the left bank for the operation of spillway No. 3.

Regarding the physical model, it allowed the simulation of the most important phenomena, namely the erosion assessments. It confirmed the discharge capacity limitation of the river downstream the existing road bridge, as anticipated by the 1D and 3D numerical models. Regarding the left bank, it allowed an indirect assessment of the locations where the flow erosion capacity in the plunge pool is stronger and lately verification of remedial measures based on a riprap protection.

As a final remark, the mixed approach considered for the hydraulic study of the Fierza dam spillway, by means of a simple 1D numerical model, a complex 3D numerical model and a traditional physical scale model, made it evident that each tool presents its own advantages and limitations, being the global result improved by the judicious combination of them all.

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