Physical and Numerical Modelling of Cerro del Águila Dam: Hydraulic and Sedimentation

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Introduction

Cerro del Águila project in Peru represents the third step of the Mantaro River major hydropower scheme cascade development. This new 510 MW capacity hydro scheme will include an 80 m high-270 m long gravity RCC dam. The 6 central blocks of the dam are equipped with mobile gates providing a total capacity of the surface spillway of around 7’000 m^3/s. The additional 6 bottom outlets increase the maximum total capacity to 12’000 m^3/s which corresponds almost to the PMF value. During the initial study, several relevant criteria related to the concept of the dam and its location were considered. First, a general investigation of the sediment yield to the reservoir due to the high erosion rate occurring in the upper sections of the river was investigated. This investigation also took into consideration the flushing works of the existing reservoir hundreds kilometres upstream of the new dam. This initial investigation showed that the annual sediment yield might vary from several hundreds of thousands to several millions of cubic meters of sediment. In order to allow adequate flushing of the new reservoir, the capacity of the bottom outlet was significantly increased and its geometry optimized. Annual stepped partial flushing is proposed to allow the transport downstream of the deposited sediments at the upper sections of the reservoir. The present paper illustrates the geometry of the Cerro del Águila dam and presents the most relevant works and results of the physical and numerical modelling of the hydraulics of the spillway and bottom outlets and the sedimentation and flushing of the reservoir.

1. Physical model investigation

The physical model of Cerro el Águila dam focused mainly on the hydraulic capacity and behaviour of the spillway and bottom outlet and on sediment flushing of the reservoir. A general model of the dam and the reservoir was built in a scale of 1:70 and another more specific larger model of two spillway bays and two bottom outlets was built in a scale of 1:40. The former aims to investigate the general hydraulic behaviour of the different organs of the dam and sedimentation process and sediment flushing, and the latter aims to provide a detailed insight on the hydraulic capacity and pressure oscillation along the spillway chute and bottom outlets during flood events. The physical modelling was carried out by Protecno S.r.l. located in Voltabarozzo-Padua, Italy, a semi private institute owned by the Italian Ministry for Infrastructure and Transport – Venice Water Authority.

1.1 Model Description

The general model of the dam and the reservoir, whose aim is to evaluate and investigate the general hydraulic behaviour of the dam and each of its elements, as well as sediment flushing from the reservoir, was built in a scale of 1:70 (Figure 1), according to Froude similarity criterion. The model reproduces approx. 1 km of the reservoir, the entire dam, with particular respect to the outlet facilities, including the radial and flap gates of the surface spillway, and 350 m of the river bed downstream the dam. The side and the bed of the reservoir and of the valley downstream the dam was reproduced in roughened mortar, while the outlet facilities was built using PVC and painted smoothed mortar, leading to proper reproduction of natural roughness, as reduced according to roughness scale ratio, defined in reference to Strickler roughness coefficient. The riverbed nearby the dam was reproduced using loose coarse gravel, to allow the investigation of jets impact area inside the plunge pool (Figure 2).

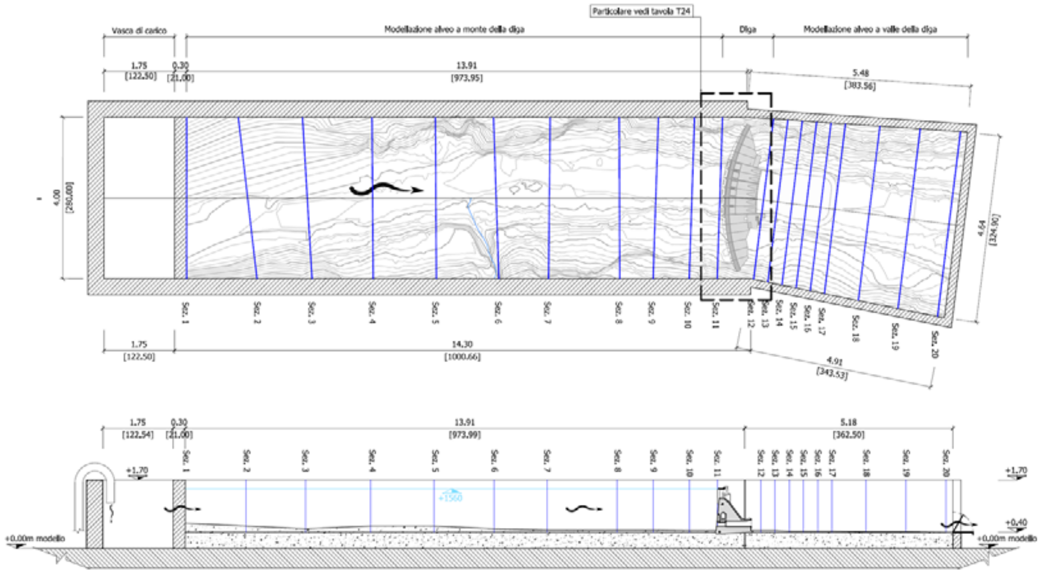

Fig. 1. Plan view (upper panel) and axial cross section (lower panel) of the physical model in scale 1: 70.

The particular model of the outlet facilities, whose aim is to allow more precise investigation of the hydraulic behaviour and capacity of the surface and bottom outlet, evaluating discharge flowing through the dam and pressure acting over the chute and inside the bottom outlet, was built in scale 1: 40 (Figure 3). The model reproduces two surface spillway bays, equipped with radial gates and two bottom outlet, equipped with 4 different sliding gates simulating 4 opening condition. In order to reproduce properly the flow conditions through the bottom outlets, the aeration system was also installed.

Fig. 2. General view of the physical model in scale 1: 70 (left) and downstream view of the 6 centrals blocks equipped with surface spillway and bottom outlets (right).

Both models are equipped with:

• manual hydrometer, in order to evaluate water level up and downstream the dam. This kind of instruments allows very precise measurement. The resolution is equal to 0.1 mm. Considering the experimental uncertainty, the measurement error can be assumed equal to 1 mm, at the model scale.

• electromagnetic discharge gauges, in order to define the discharge flowing through the model and across the outlet facilities of the dam. The uncertainty in the discharge measurements can be considered equal to 1 % of the maximum discharge required, that is 3 l/s, at the model scale.

Fig. 3. 1:40 scale model of the surface spillways (left) equipped with radial gates (centre) and bottom outlet.

For the model built in scale 1: 40, pressure reading on the spillway chute have been performed using 14 water intakes, connected to a small glass pipe, placed on a flat plastic plate. With zero discharge flowing, each pipe shows the elevation of each intake; the water level in static condition (no discharge) is the reference point to measure under or overpressure in dynamic condition, when water is flowing along the chute. The pressure acting into the bottom outlet was measured via 9 absolute electronic pressure gauges (Figure 4).

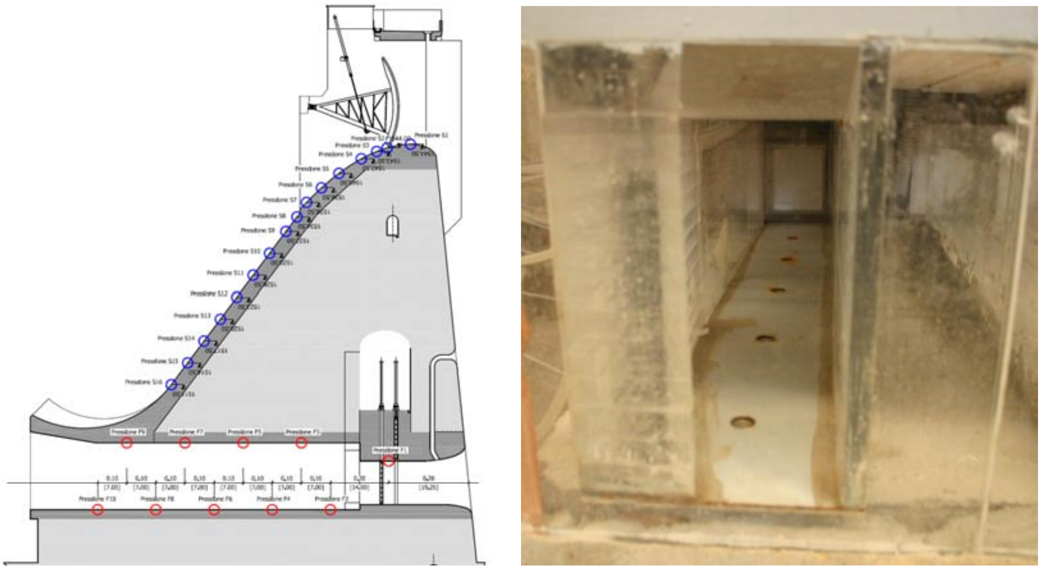

Fig. 4. Pressure gauge equipment of the 1:40 scale model of the spillway chute (left) and the bottom outlet (right).

1.2 Hydraulic tests and results

In order to evaluate the general behaviour of the dam, seven tests, reproducing different opening configurations for the surface and bottom outlets and different discharge conditions, were performed. During each test, water level up and downstream the dam and discharge flowing through the model was measured. Also, the general hydraulic behaviour of the model has been monitored.

Fig. 5. Hydraulic preliminary tests. reproduction of the PMF (approx. 12’000 m³/s)

The general behaviour of the dam appears completely satisfying (Figure 5). The discharge capacity of the surface spillway (Figure 6) and of the bottom outlets results able to evacuate the PMF (approx. 12’000 m³/s). Approach conditions upstream the dam are completely satisfying: no vortex can be seen in the reservoir when discharge approaches the surface spillway. The weirs shows satisfying behaviour: along the higher part of the chute the flow is always regular and only for tests characterized by higher discharge the flow appears turbulent. Along the chute the flow is regular, showing some aeration under the flow in the higher part. In the lower part, where flows coming from different blocks meet, a shockwave is located. The side walls height appears suitable to restrain the discharge flowing across the surface spillway.

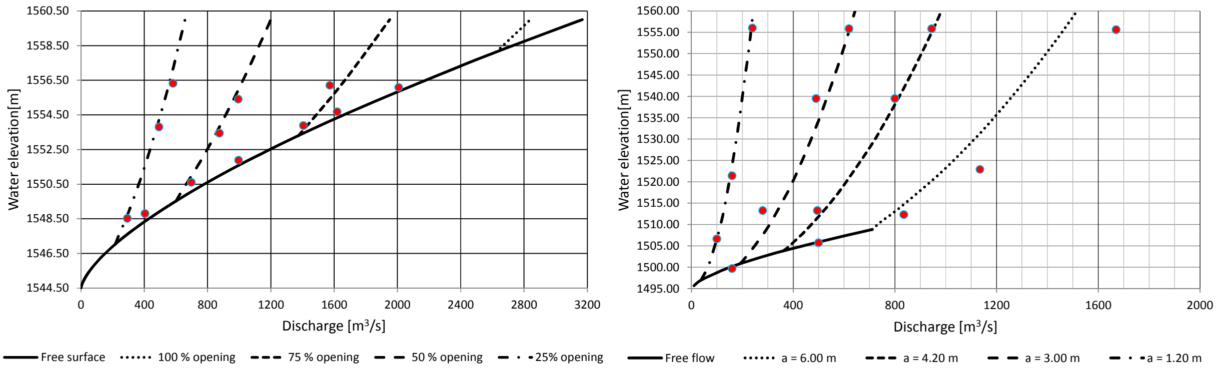

Fig. 6. Discharge capacity for surface spillway (left) and bottom outlet (right).

During each test the main dissipation take place inside the plunge pool, even for higher discharges. In general also bottom jet is located into the plunge pool; only for higher discharges it extends outside this area, reaching a distance of 200 m approximately from dam axes. In order to reduce bottom excavation into the plunge pool, the outlet structures of the bottom outlets equipping blocks n. 1 and 2 have been modified. The original shape of this structures concurred to concentrate the two jets out of these structure in one single jet, characterized by higher energy and leading to stronger excavation in the central area of the plunge pool. To reduce this phenomenon, the diverting structure have been removed. The modified layout doesn’t modify the discharge capacity, but let the excavation in the central area of the plunge pool becomes smaller, reducing the accentuated V shape.

In order to evaluate the discharge capacity of both the surface spillway and of the bottom outlet, 20 different configurations have been tested for the different gates, different gates opening configuration and different gates opening grade; 37 tests have been performed on the model scale 1:70 and 26 on the model scale 1:40.
The discharge capacity for the surface spillway results slightly smaller than the theoretical discharge capacity calculated by the designer (2-3% smaller than the theoretical calculation). This phenomenon can be understood considering the mutual influence among each weir, expressing in a reduction of the water elevation upstream the spillway, due to call effect. Concerning the discharge capacity of the bottom outlets, the experimental values result a slightly higher than the theoretical one calculated by the designer (about 4% higher than the theoretical calculation). This can be explained considering the difficult theoretical evaluation of the head – loss at bottom outlet entrance.

Fig. 7. Jet trajectories (left), water elevation along the surface spillway (centre), pressure acting over the chute (right).

During the tests performed in the model built in scale 1: 40 also water elevation along the piers and the side walls, pressure acting over the chute and inside the bottom outlet and jet trajectories have been measured. None of these parameters showed unexpected results (Figure 7).

1.2 Sediment deposition and flushing

The main features of the downscaled sediment represent the same characteristics of the sediment found along the banks of Mantaro River. According to this information, the mean grain size has been considered equal to 1.000 mm. The specific weight of sediment is equal to 2650 daN/m^3. In order to reproduce sediment flushing due to gate opening and sediment removal from the upstream reservoir, mono granular silica sand has been used. The sand used for the simulation is characterized by really small grains. Exploiting finer materials would lead to insignificant results, due to cohesion. The behaviour of this material has been evaluated and compared to the real sediment according to Van Rijn theory (1984), for the different hydraulic conditions of the tests (Figure 8).

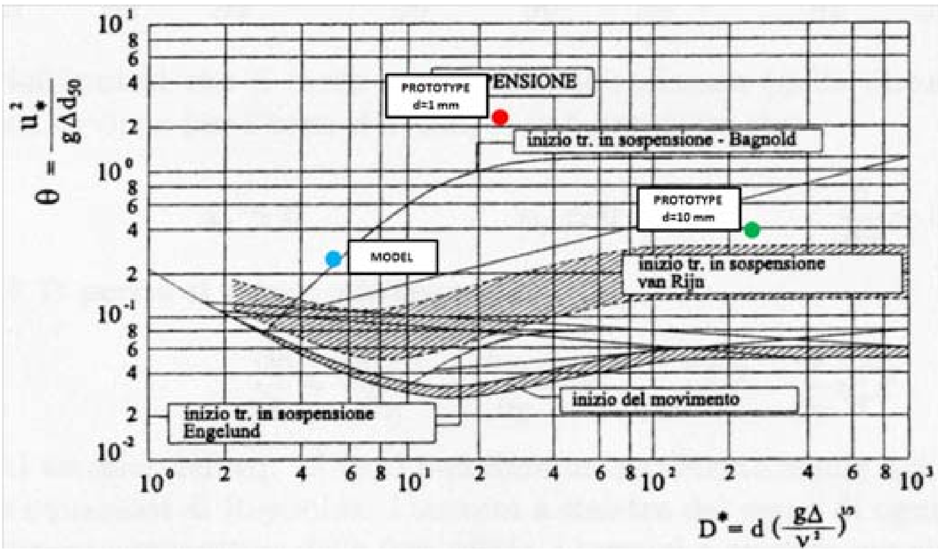

Fig. 8. Behaviour of model, prototype and equivalent sand, according to Van Rijn and shields Theories.

The sand used for the model well represents the behaviour of the natural sediment. Even if it shows smaller disposition toward movement, in every case the transport processes are properly reproduced, leading to significant but underestimated results. Since the used sand does not reproduce exactly the mean diameter of natural grains, according to Shield theory the effective size of the material represented in the prototype by the sand used in the model has been evaluated. The critical shear stress in the model is equal to 0.14 N/m², while the equivalent value for the prototype is equal to 10.08 N/m², so the equivalent mean diameter of the grains in the prototype is approx. 10 mm. Two kinds of tests have been performed to analyse the sediment removal from the bottom outlets entrance and from the upstream reservoir. The first one reproduces the opening of 2, 4 and 6 bottom outlets gates, in order to check if this manoeuvre is able to remove sediment from bottom outlet entrance. These tests have been performed with an opening grade of the gates set to 50% and a constant value of the reservoir water level (1546 and 1556 m asl). The second one reproduces the removal of sediment from the bottom of the upstream reservoir, with different constant discharges (approx. 450 – 600 – 700 – 800 m^3/s) flowing across the bottom outlets, with the gates completely opened. These tests have been realized with 4 and 6 gates opened.

Fig. 9. Flushed area near bottom outlets after sudden gate opening.

For each simulation, the bottom of the reservoir have been filled with non-cohesive sand, till the height of 1’503 m asl. After each test, the flushed sediment have been evaluated. Tests reproducing bottom outlet entrance sudden opening shows that in every conditions this manoeuvre leads to complete removal of sediment nearby the entrance. For each outlet, the cleared area reaches an extension of approx. 15x 20 m; its minimum elevation is always lower than outlet bottom. No particular differences have been observed between tests with different water level in the reservoir.

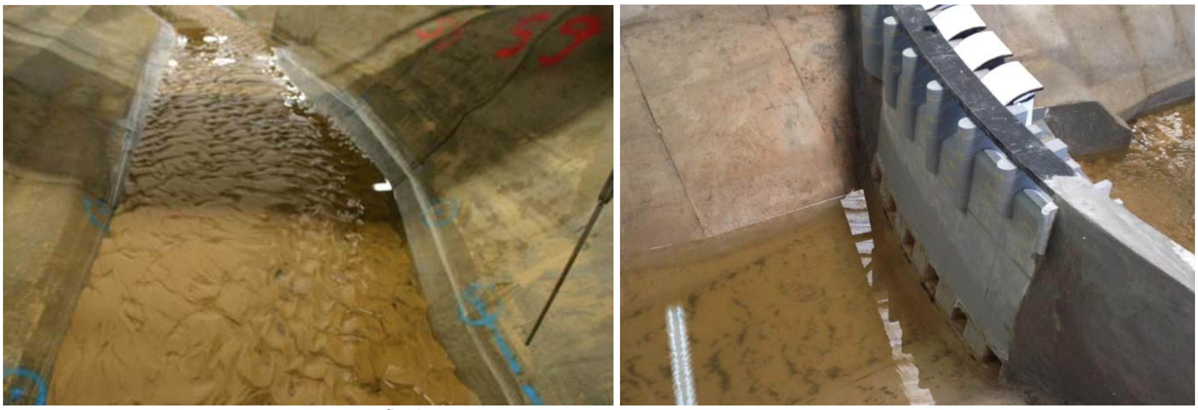

Fig. 10. Reservoir flushing tests.

Tests reproducing sediment flushing from the bottom of upstream reservoir section show good results for simulations performed with four and six outlets. With four gates opened, the river moves approx. 250’000 m^3 of settled sediment in approx. 8 days (prototype scale). This means a sediment discharge of approx. 0.35 m^3/s. The mean thickness of the eroded layer is of approx. 3 m over the whole investigated area. With six gates opened, the river moves a little more sediments. The removed volume is equal to approx.320’000 m^3, a value that seems to be independent from the discharge flowing across the model. This means a sediment discharge of approx. 0.44 m^3/s. The mean thickness of the eroded layer is of approx. 4 m over the whole investigated area.

2. Numerical modelling of sedimentation and flushing procedure

In order to estimate the sedimentation impact in the Cerro del Águila lake and the need of flushing (frequency, duration and flow), a numerical model has been studied. Reproducing the entire lake of 11 km, two 2D models (BASEMENT and CE-QUAL) have been developed in order to establish suspended load sediment deposition along the lake and the hydraulic uniform flow through the cross section. This part of the paper emphasizes on the 1D model (HEC-RAS) and its results of total sediment deposition as well as on the lumped model (RESCON) and the requirement of total flushing.

2.1 Model description

The 1D HEC-RAS model is used to perform a mobile bed sediment transport analysis in the whole reservoir for several long term scenarios. Current sediment capabilities in HEC-RAS (Figure 11) are based on a quasi-unsteady hydraulic model. The quasi-unsteady approach approximates a flow hydrograph by a series of steady flow profiles associated with corresponding flow durations. The sediment transport equations are then solved for each time step.

2.2 Geometry and parameters of the model

The model is built with sections at a distance of 10 m over the 10.2 km length of the reservoir. The Manning roughness coefficient, n, is equal to 0.05 $s/m^{1/3}$. In HEC-RAS model, a transport function model needs to be selected by the user. Sediment transport results are strongly dependent on which transport function is selected. Usually when measurements are available, the proper function can be choosen in the model calibration step. In the present study, several functions are tested. Considering the range of assumptions, hydraulic conditions, and grain sizes, the Toffaleti (Tofaletti, 1968) function is selected. Tofaletti appears to be the most addapted function for modeling suspended load and for the range of grading.

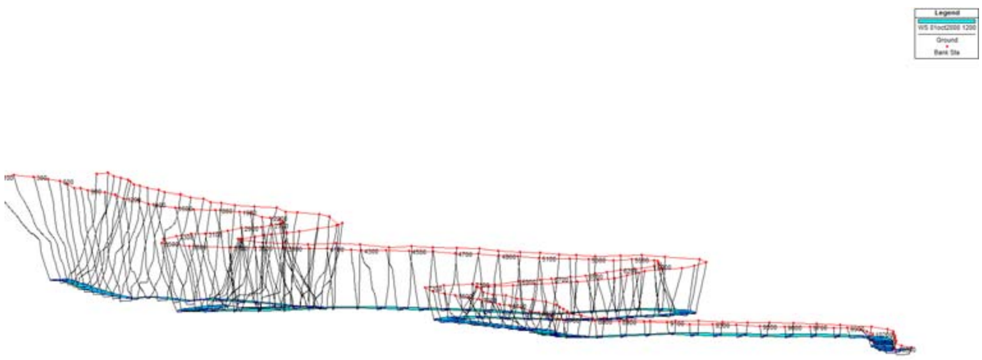

Fig. 11. Hec-Ras model of the entire Reservoir.

2.3 Boundary conditions and transport function

To define the bed load transport capacity as a function of water discharge, different relationships such as modified Meyer-Peter & Müller (MPM) (Meyer-Peter & Müller, 1948), modified MPM (Wong & Parker, 2005) and Smart-Jäggi (1983) are compared. For this purpose a representative section of the river upstream of the reservoir is selected and the solid transport capacity is calculated over a year with a daily time step. The water flow is the mean daily flow. The total bed load sediment volume in one year is then calculated and compared with the 1.7 Mm^3 of expected bed load. The modified MPM method by Parker gives a total annual bed load of 1.69 Mm^3 and is therefore chosen. In addition, the MPM method is the best suited for the range of slopes in the river and the sediment grading.

2.4 Reservoir sedimentation results

The HEC-RAS model is firstly used to model sediment transport over a mean year. For this purpose the model is run two times, once for the bed load and once for the suspended load. The upstream flow boundary conditions is the flow series with daily values. At downstream, a fixed water elevation equal to the normal water elevation of the dam is considered. The upstream sediment boundary condition is specified as sediment load series for the uppermost section.

As it can be expected, the bed load forms a delta at the entrance of the reservoir. Due to the grain size and the low velocities in the reservoir, the delta cannot move forward downstream and the material accumulates at the upstream limit of the model. However, the suspended material remains on suspension while entering the reservoir and settles down in its middle.

The simulations start on October 1st and end on September 30th of the next year. As it can be seen in Figure 12 the deposition starts on January and continues over the wet season until end of April. From April to September, during the dry season, the sediment yield is negligible and there is no more deposition in the upper part of the reservoir. However, minor erosions can occur on the upper part of the deposited delta and produce a small amount of material moving downstream.

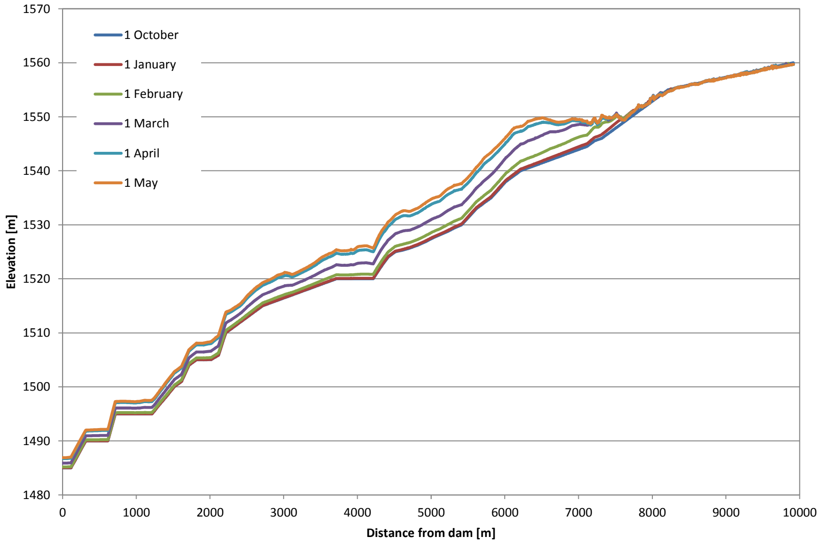

Fig. 12. Suspended load deposition during one year (reservoir normal water elevation 1556 m asl).

To better illustrate the same results, a plan view of deposition zones is shown in Figure 13. No sedimentation happens for the first 2'300 m of reservoir upstream. Then the deposited layer height increase to 8.9 meters at a distance of about 6'000 m from the dam. The deposition thickness reduces then to 2 m behind the dam.

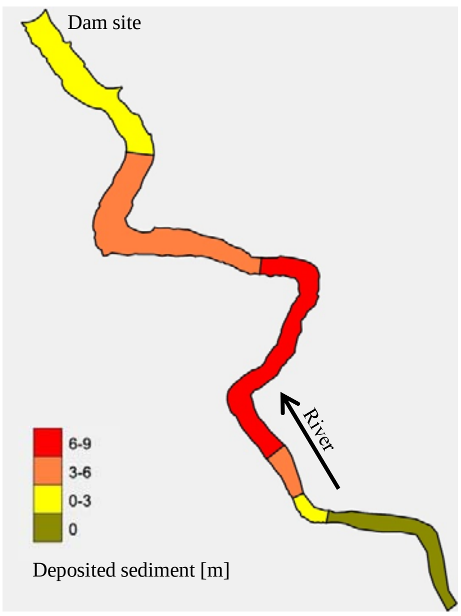

Fig. 13. Suspended load deposition zones after one year.

For the long term deposition, a simulation is carried out over five years. The results are plotted in Figure 4. It is shown that the deposited delta moves on downstream by 2 km each year. A removal measure then seems crucial as the reservoir loses more than half of its capacity in only five years. Figure 14 shows the bed level evolution during 5 years just behind the dam (5 m upstream). As it is shown, during wet seasons high amounts of sediments are deposited behind the dam, whereas for dry seasons there is no deposition. The annual sediment deposition at this section is increased each year comparing to the previous one as the delta approaches the dam. The deposition height at fifth year (about 4 meters) is approximately two times more important than that of the first year (about 2 meters).

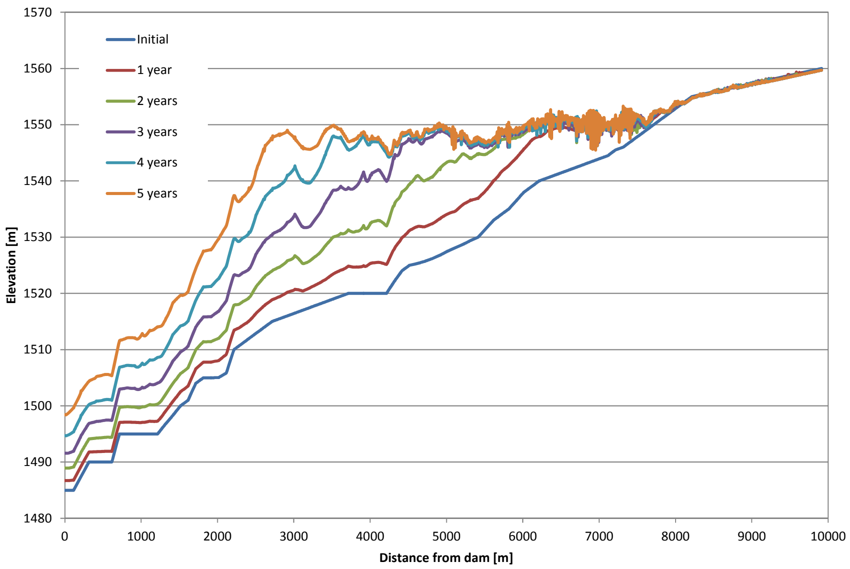

Fig. 14. Suspended load deposition during five years (reservoir normal water elevation 1556 m asl).

2.5 Lumped model (reservoir flushing results)

RESCON model is a tool that is used principally for pre-feasibility studies for dam construction projects. The basis of the technical model for flushing is Atkins work which quantifies aspects of reservoirs that are likely to be successful in flushing at complete drawdown. The capacities are calculated using simplified reservoir geometry. Several flushing conditions are calculated using the RESCON program. Simulations with Flow 3D program show that the pressurized flushing is useful only to remove the sediment clogged at the bottom outlet entrance and washes out a cone which is extended between 15 to 20 m upstream the bottom outlet. Flow 3D results also show a transition from pressurized to free surface flow at a flushing elevation of about 1505 m asl. Therefore, to guarantee the flushing efficiency, a free surface flow has to be established. The considered flushing elevation then covers the range of elevations between 1498 and 1505 m asl. As a general fact, for a given discharge, the lower the elevation is, the more successful any flushing operation will be. The hydraulic capacity of the outlet must be sufficient enough to maintain the reservoir at a constant level during the flushing period and with minor fluctuations in level in order to activate sediment movement. In addition, a flushing water volume of at least 10% of the annual runoff should be expected.

Flushing discharges of at least twice the mean annual flow are required for an efficient flushing. In the case of Cerro del Águila dam, the mean annual flow is about 270 m^3/s. For an annual flushing, the maximum expected discharge is the annual flood discharge, i.e. 1’400 m^3/s. As such, a range of flushing discharges between 540 to 1’400 m^3/s are the most reasonable. However, the 6 bottom outlets in their initial dimensions may not have the capacity for passing an annual flood as free surface flow.

Some selected results are shown in Table 1, where calculations are done for annual flushing. The duration of flushing is the time after complete drawdown of the reservoir. The flushing discharge for each reservoir elevation is calculated. In RESCON model, a safety factor of 2 is considered. The flushed sediment mass and volume is presented for each case. Considering a discharge larger than two times the mean annual flow (540 m^3/s), total flushing is only efficient for a corresponding flushing elevations more than 1’500 m asl. The concentration downstream the dam can also be calculated as a function of flushed sediment volume as well as duration and discharge of flushing. Some norms do not accept high solid concentration in the river during a long time. In this project the limits have to be checked in Peruvian norms.

Flushing drawdown level	Flushing discharge	Sediment vol flushed in one day	Concentration downstream	Needed time after complete drawdown
m asl	m^3/s	Mm^3	ml/l	day
1498	212	0.9	47	6.0
1499	338	1.8	61	3.0
1500	485	3.1	73	2.0
1501	652	4.8	85	1.0
1502	837	6.9	95	1.0
1503	1039	9.4	105	0.5
1504	1258	12.4	114	0.5
1505	1492	15.8	123	0.5

Table 1: RESCON results for annual flushing.

According to the results, the required flushing time varies between 8 hours and 6 days. Efficiency of the flushing increases by the increase of flow discharge. According to the concentration values which can be estimated as quite high and comparing to others reservoir flushing experience, an annual flushing duration of minimum 1 to 2 days can be recommended with a flow equivalent to the annual flood event (1’400 m^3/s).

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Sante Bonanni obtained his degree in hydraulic engineering at the University of Rome. At present he works as a chief design engineer and expert in hydraulic works at Astaldi S.p.A., Engineering and Design Department. He is responsible for the design of hydroelectric power plant worldwide with particular reference to dams. He followed the design of the Cerro del Águila project since the initial tendering until the final draft with the designer Lombardi SA.

Philippe Heller accomplished his civil engineering degree and later obtained his Ph.D. in hydraulics at the Ecole polytechnique fédérale de Lausanne (EPFL), Laboratory of Hydraulic Constructions in 2007. He assumes the role of director of the engineering firm e-dric SA, a company based in Lausanne and specialized in design of hydraulic structures and numerical modelling in the fields of hydraulic, hydrology, and coastal. He supervised the design activities related to the numerical modelling of the sedimentation process and flushing of the reservoir of Cerro del Águila.

Matteo Volpato accomplished his civil engineering degree at the University of Padova in 2005 and since then he assumes the role of chief project manager at Protecno S.r.l. His main activities concern hydraulic works construction and supervision, hydraulic modelling and design. He designed and supervised the construction of the hydraulic models of Cerro del Águila dam and completed the analysis and interpretation of all the results.