

ANALYSIS OF SEDIMENTATION AND FLUSHING INTO THE RESERVOIR PAUTE-CARDENILLO

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Abstract

The study analyzes the expected changes in the Paute River in Ecuador as a result of the construction of the Paute-Cardenillo Dam. Given that the project must remain viable throughout its useful life, the operational rules at the reservoir are required to include sedimentation effects. Four complementary procedures are used: empirical formulae, one-dimensional simulations (sedimentation), two-dimensional simulations (flushing) and three-dimensional simulations (detail of the bottom outlet).

Keywords: Flushing; Reservoir; Sedimentation; Simulation

1. Main characteristics of the project

The study zone is situated in the Paute River basin in Ecuador to 23 km downstream from the Amaluza Dam. The area to be analyzed is of 275 km² of draining surface and the average slope of the river reach is 0.05 (Figure 1).

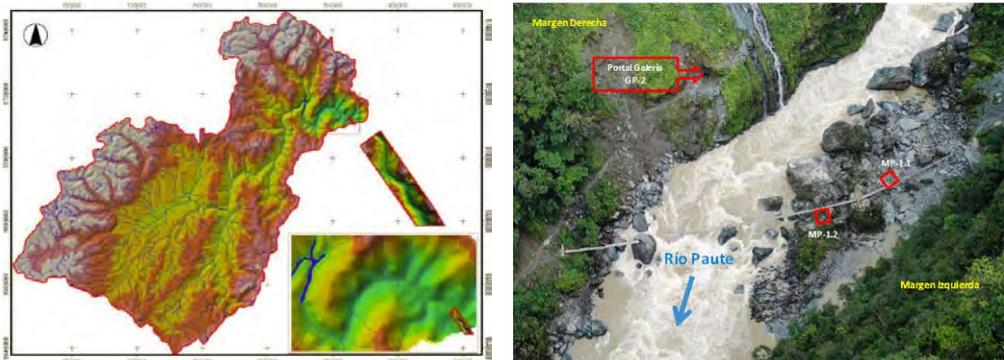


Figure 1. (a) Zone of study in Paute River basin. (b) Paute River.

Paute-Cardenillo (owned by Celec Ep-Hidropaute) is a double curvature arch dam with a maximum height of 135 m to the foundations. The top level is located at 926 meters. The reservoir has a length of 2.98 km, with normal maximum water level being located at 924 meters. This dam will integrate the National Electric System of Ecuador with a total electricity installed capacity of 600 MW which will produce 13000 GWh per year.

Figure 2 shows the sieve curves obtained at three sites of the river and the mean curve used in the calculations.

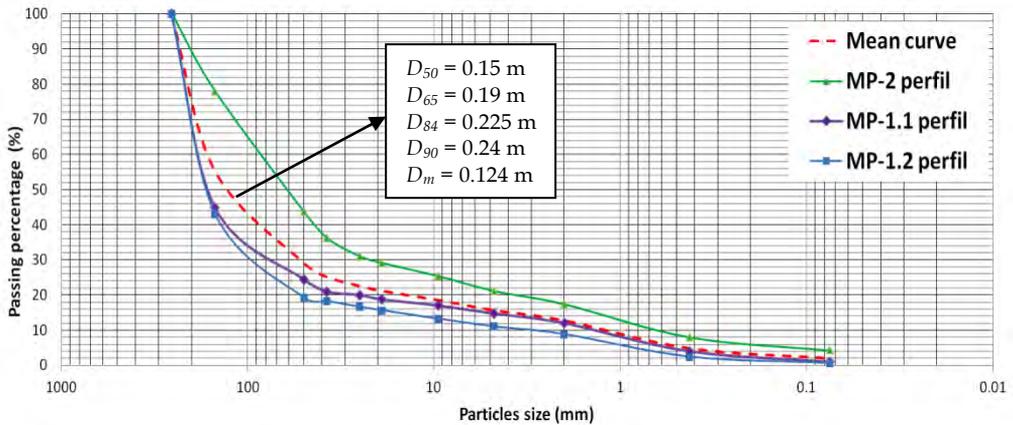


Figure 2. Sieve curves of three sites of the river and the sieve mean curve.

The total bed load (excluding wash load) was determined as 1.75 Mm³/year and the maximum volume of the reservoir 12.33 Mm³. In order to prevent the deposition of sediments into the reservoir, periodic discharges of bottom outlet or "flushing" have been proposed. These operations should be able to remove the sediments, avoiding the advance of the delta from the tail of the reservoir.

Initial studies indicate that the minimum flow evacuated by the bottom outlet to achieve an efficient flushing should be at least twice the annual mean flow ($Q_{ma}=136.3 \text{ m}^3/\text{s}$). For the safe side, a flow of 408.9 m³/s ($3Q_{ma}$) was adopted.

2. Sediment transport formulae

Sediment transport may be divided into the following: wash load (very fine material transported in suspension), and total bed transport (bed sediment transported and/or in suspension, depending on the sediment size and flow velocity). The main properties of sediment and its transport are the following: the particle size, shape, density, sedimentation velocity, porosity and concentration.

2.1 Estimate of the Manning resistance coefficient

The calculation of the flow characteristics depends mainly on the resistance coefficient, hydraulic radius and longitudinal slope. Following the methodology applied in Castillo *et al.*, 2009, four aspects were checked to determine hydraulic characteristics of the flow: macro roughness, bed form resistance, hyper concentrated flow, and bed armoring phenomenon.

Ten formulae were applied for estimation of the roughness coefficient: Strickler, 1923; Limerinos, 1970; Jarret, 1984; Bathurst, 1984; van Rijn, 1987; Fuentes and Aguirre-Pe, 1991; García-Flores, 1996; Grant, 1997; Fuentes and Aguirre-Pe, 2000; and Bathurst, 2002. These

formulae are calculated by coupling iteratively the hydraulic characteristics with the sediment transport.

A macro roughness behavior may be identified in all the flows analyzed, which also present the armoring phenomenon. This leads to a significant increase in the Manning coefficients. The calculation of these coefficients was carried out in a section type, through an iterative procedure by using the formulation of Fuentes and Aguirre-Pe *et al.*, 2000. Figure 3 shows the Manning coefficients for the flow rates.

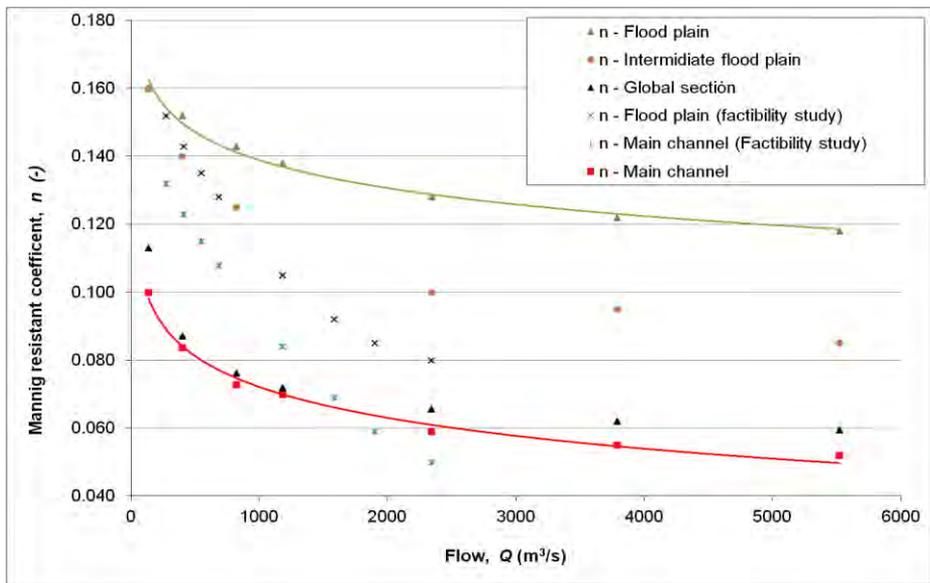


Figure 3. Manning resistance coefficients in the main channel and floodplain.

2.2 Estimation of sediment transport

Fourteen formulations of sediment transport capacity were used: Meyer-Peter and Müller, 1948; Einstein and Brown, 1950; Einstein and Barbarrosa, 1952; Colby, 1964; Engelund and Hansen, 1967; Yang, C.T., 1976; Parker *et al.*, 1982; Smart and Jaeggi, 1983; Mizuyama and Shimohigashi, 1985; van Rijn, 1987; Bathurst *et al.*, 1987; Ackers and White, 1990; Aguirre-Pe *et al.*, 2000; and Yang, S., 2005. From these, the formulations that fell within a range of the mean value ± 1 standard deviation were selected. Figure 4 indicates that the transport capacity could vary between 1 and 100 t/s, if the mean values of the analyzed reach are considered. However, these values are reduced between 0.5 and 10 t/s when the river complete reach is considered (erosion and sedimentation processes are simulated). Finally, the net sediment transport in dam site was only 0.2 t/s.

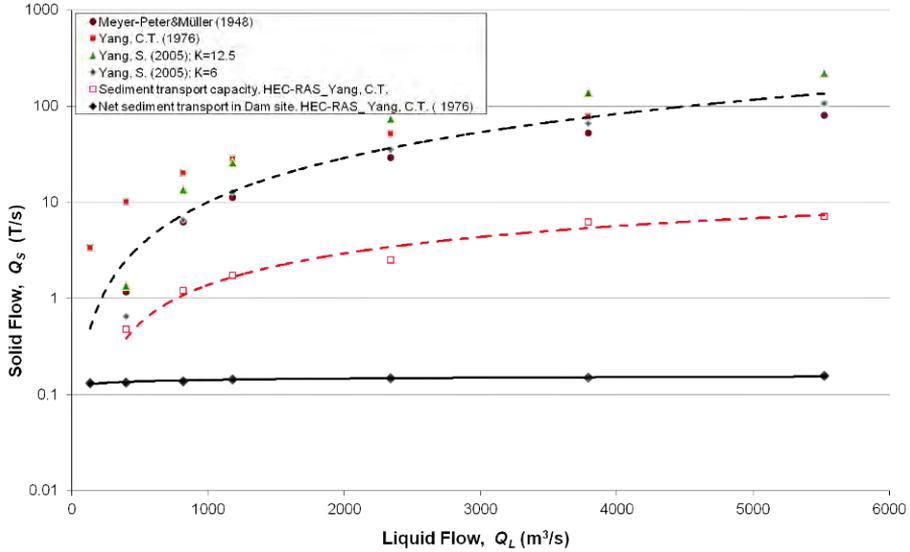


Figure 4. Sediment transport capacity (reach mean values), sediment transport (simulation of river complete reach) and net sediment transport in dam site.

3. Numerical simulations

The bed level change Z_b can be calculated from the overall mass balance equation for bed load sediment (Exner equation):

$$(1-p) \frac{\partial Z_b}{\partial t} + \frac{\partial Q_{bs}}{\partial s} + \frac{\partial Q_{bn}}{\partial n} = 0 \quad [1]$$

where p is porosity of the bed material; Q_{bs} and Q_{bn} are the bed load flux in the main flow direction s and in the cross flow direction n . They are calculated from the non-equilibrium bed load equation:

$$\frac{\partial (Q_b a_{bs})}{\partial s} + \frac{\partial (Q_b a_{bn})}{\partial n} = -\frac{1}{L_s} (Q_b - Q_e) \quad [2]$$

where a_{bs} and a_{bn} are the cosines of the direction vector that determines the components of the bed load transport in the s and n directions, respectively. All non-equilibrium effects are expressed through the model on the right hand side, assuming the effects to be proportional to the difference between non-equilibrium bed load Q_b and equilibrium bed load Q_e and related to the non-equilibrium adaptation length L_s .

3.1 Reservoir sedimentation

The time required for sediment level to reach the height of the bottom outlets (elevation 827 m) operating at reservoir levels was analyzed. Simulations were carried out with the one-dimensional HEC-RAS 4.1 program which employs a continuity equation of sediment.

The input flows are the annual mean flow ($Q_{ma}=136.3 \text{ m}^3/\text{s}$) equally distributed in the first 12 km and the incorporation (2.44 km upstream from the Paute-Cardenillo Dam) of the annual mean discharge flow of the Sopladora hydroelectric power plant ($Q_{ma_sop}= 209.0 \text{ m}^3/\text{s}$).

The suspended sediment concentration at the inlet section was $0.258 \text{ kg}/\text{m}^3$. This value is similar to the mean concentration at the Sopladora hydroelectric power plant. The sediment characteristic diameter in the dam emplacement was $D_{50} = 0.150 \text{ m}$. The sediment transport was calculated by considering the Meyer-Peter and Müller, 1948, and Yang C. T., 1976, formulae. In addition, a reference hydrograph (three times the average annual flow), and hydrographs with different return periods, were also considered.

Table 2 shows the volume of sediments in the reservoir obtained when the bottom outlet level was reached. The two transport equations, as well as various water levels in the reservoir were examined. According to the results, the volume of sediment in the reservoir rises with the increasing of the water level in the reservoir, and requires a longer duration to reach the bottom outlet elevation.

Table 2. Time required and volume of sediment when the bottom outlets are reached.

RESERVOIR ELEVATION	YANG		MEYER-PETER & MÜLLER	
	REQUIERED TIME (YEARS)	SEDIMENT VOLUME (hm^3)	REQUIERED TIME (YEARS)	SEDIMENT VOLUME (hm^3)
860	0.33	0.65	0.33	1.47
892	5.10	2.77	2.58	3.86
918	12.90	6.07	8.80	7.34
920	13.60	6.33	9.50	7.64
924	14.80	6.62	10.90	8.97

The least favorable condition (the first one in which the sediment reaches the elevation of 827 m) was obtained with the expression of Meyer-Peter and Müller and the level of the reservoir located at 860 m, requiring a time of three months and 27 days.

3.2 Flushing simulation

3.2.1 Two-dimensional simulation

Since the one-dimensional model was unable to reproduce the regressive erosion of sediment in the reservoir, the flushing process was analyzed by using the Iber two-dimensional program. Iber can be divided in three modules: hydrodynamic, turbulence and sediment transport. The program uses triangular or quadrilateral elements in an unstructured mesh and finite volume scheme. The hydrodynamic module solves shallow water equations (2-D Saint-Venant equations). Diverse turbulence models with various levels of complexity can be used. The sediment transport module solves the transport equations by the Meyer-Peter & Müller and van Rijn expressions.

According to the operational rules of the Paute-Cardenillo Dam, the evolution of the flushing is studied over a continuous period of 72 hours. The initial condition of sedimentation profile (the lower level of the bottom outlet) was the obtained with the HEC-RAS simulation (1.47 hm^3 of sediment). The input flow was three times the annual average flow ($408.9 \text{ m}^3/\text{s}$). The

suspended sediment concentration at the inlet section was 0.258 kg/m^3 . This value was obtained from the flow drained by the upstream Sopladora hydroelectric power plant ($209 \text{ m}^3/\text{s}$) which had a mean concentration of 0.250 kg/m^3 . The initial water level at the reservoir was 860 m. Effective flushing was observed during the operation of the bottom outlets. Figure 5 shows the transversal profiles of the reservoir bottom before and after the flushing operation.

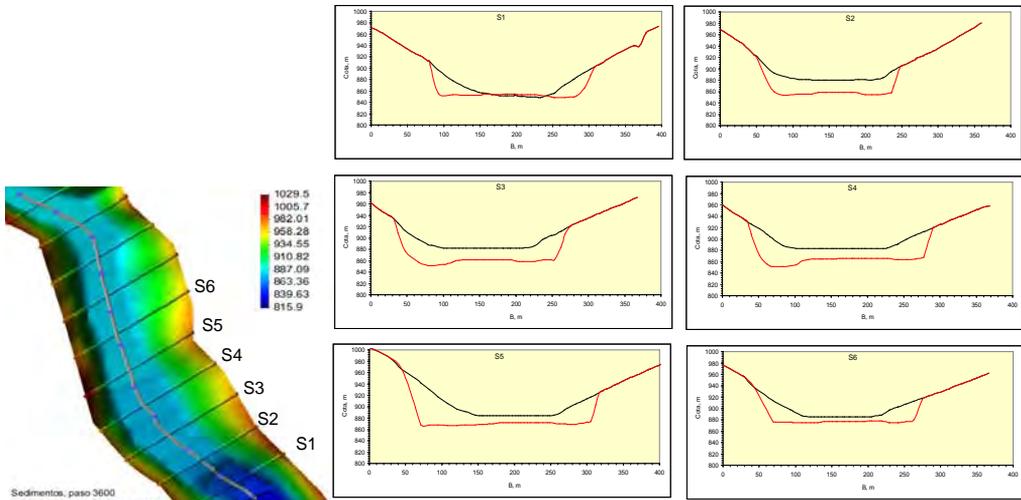


Figure 5. Level of sediments before and after a flushing period of 72 hours.

Two scenarios were considered: Initial water level at reservoir located at elevation 860 and 918 m. The erosion showed maximum values of 21 meters after 72 hours of washing in the first stage and approximately 45 meters in the second stage. These maximum values were reached at a distance of 100-200 meters upstream from the dam, where the maximum velocities occurred.

After a flushing period of 72 hours that involved a water flow of $408.9 \text{ m}^3/\text{s}$, the sediment volume removed by the bottom outlets in each scenarios were respectively 1.77 hm^3 and 3.51 hm^3 . In the first scenario, 100% of the sediments from the reservoir were swept, while in the second stage 81% were removed. These differences are due to the initial sedimentation volumes existing in each case. Considering the initial water level at the reservoir of 860 m, Figure 6 shows the relationship between the wash volume, the time, and the evolution of sediment removed through the bottom outlets. After a flushing period of 72 hours, a sediment volume equivalent to 1.77 hm^3 may be mobilized (value slightly greater than the initial deposition of 1.47 hm^3). During the initial emptying of the reservoir, higher rates of sediment transport appeared ($23 \text{ m}^3/\text{s}$ of sediments). Due to the occlusion of the outlets by the presence of a strong sediment load, the sediment transport falls below $3 \text{ m}^3/\text{s}$, and later continues with a rate near to $7 \text{ m}^3/\text{s}$. Except in the initial instants, the bottom outlets work in free surface conditions.

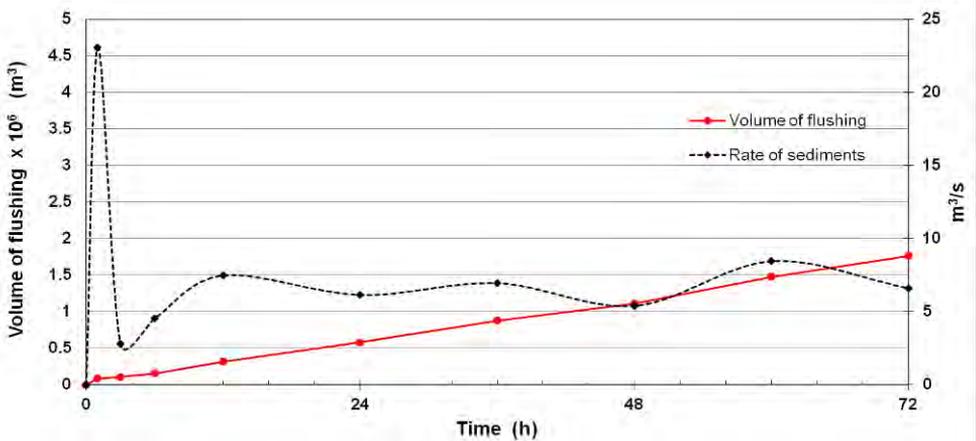


Figure 6. Evolution of the volume of sediments and the sediments transport through the bottom outlets during the flushing operation.

3.2.2 Three-dimensional simulation

The computational fluid dynamics (CFD) program FLOW-3D was used. The program uses hexahedral elements in a structured mesh and finite volume scheme. The Navier-Stokes equations are discretized by finite differences. It incorporates various turbulence models, a sediment transport model and an empirical model bed erosion (Guo, 2002; Mastbergen and Von den Berg, 2003; Brethour and Burnham, 2011), together with a method for calculating the free surface of the fluid without solving the air component (Hirt and Nichols, 1981).

The bed load transport is calculated by using the Meyer-Peter & Müller, 1948, and Van Rijn, 1986, expressions.

Due to the high-capacity equipment and long simulation times to calculate the flushing of all the reservoir, the detailed operation of the bottom outlets was analyzed. The initial conditions of sedimentation was obtained with the HEC-RAS program. The inlet boundary was situated 150 m upstream the dam.

Figure 7a shows the velocity vectors of the flow passing through the bottom outlets. Figure 7b shows the discharge flow at each bottom outlet, together with the total flow discharged and the mean flow discharged by each conduit for the first 3000 s of simulation. Outlets worked in a pressured and unsteady regime at the initial emptying of the reservoir, reaching a discharge near 1000 m³/s. After the steady regime was reached (around 230 s of simulation), there was a free surface flow and the discharged flow was the expected (408.90 m³/s during the flushing operation). The two outlets on the left side obtained a higher flow than the expected rate in the design, while the two outlets on the right worked with a lower flow than expected.

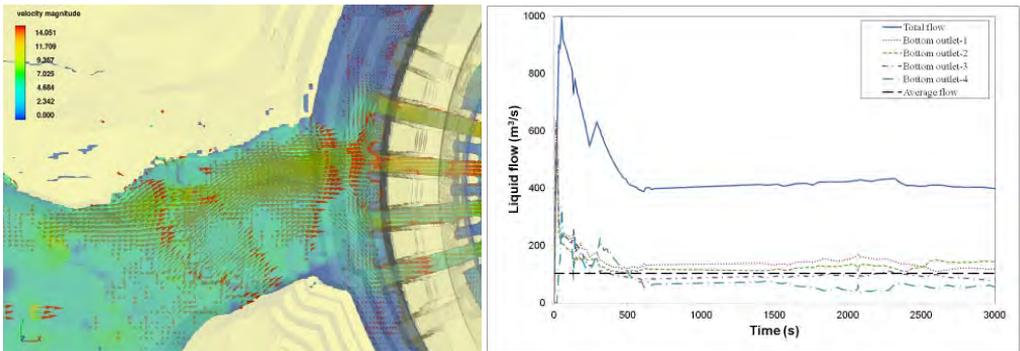


Figure 7. a) Velocity vectors through the bottom outlets during the flushing operation; b) Flow discharged for each duct.

Figure 8 shows the erosion of sediment in the reservoir after 1000 seconds of operation. Although the operation time was small, the sediment removed in the three-dimensional operation was significant. The regressive channel in the delta of sediments is clearly visible. These results match with the values observed in the two-dimensional simulation.

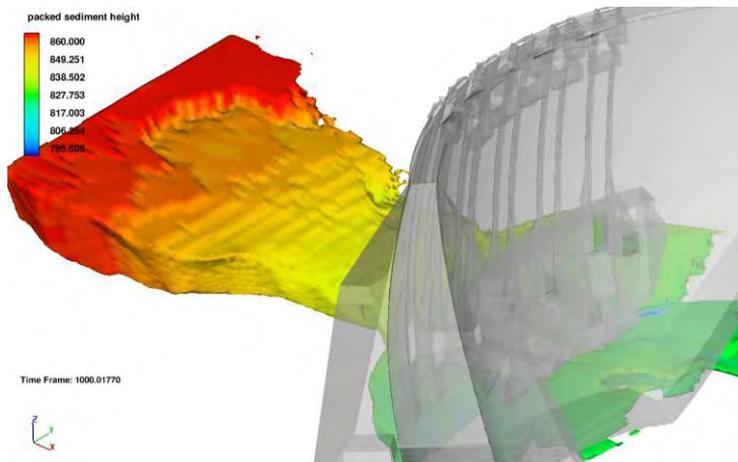


Figure 8. Evolution of the sediments delta in the three-dimensional simulation.

Figure 9 shows the lateral view of the sediment in the reservoir in the initial condition and after 5000 seconds of operation. Although only a part of the reservoir was simulated, the reduction of the sediments can be observed. The detail of the effective flushing is in agreement with the Iber simulations. Considering the complete reservoir, the regression of the delta should be slower due to the great amount of sediments.

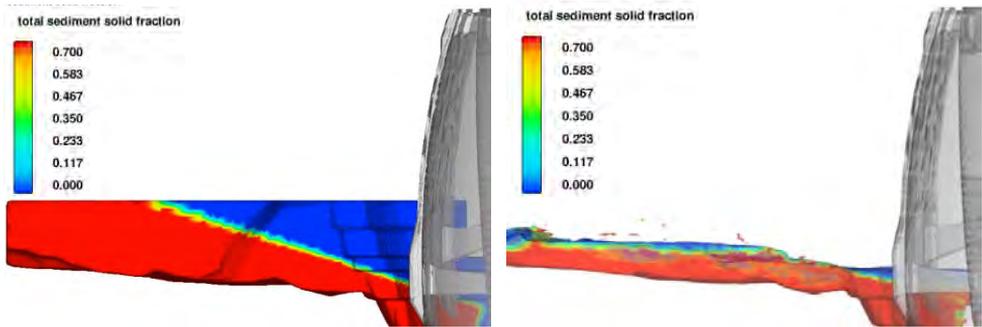


Figure 9. Lateral view of the of the sediments delta in the three-dimensional simulation: a) Initial condition; b) After 5000 s.

4. Conclusions

In this paper, the complex phenomenon of flushing has been analyzed by using four interrelated methodologies: empirical formulations, one-dimensional simulations, two-dimensional simulations and three-dimensional simulations. Empirical simulations constitute an upper envelope of the sediment transport capacity. This procedure allowed an estimate of the coefficients of resistance or Manning roughness applied in the numerical simulations.

Due to the time period (one year) required to analyzing the sedimentation process in the reservoir, and the length of the reach (23.128 km), simulations were carried out with a one-dimensional program (two and three-dimensional programs need high-capacity equipment and long simulation times). Flushing operation was simulated with two-dimensional (Iber) and three-dimensional (FLOW-3D) programs. For 72 hours of flushing simulation, the Iber program required near 24 hours (using one processor Intel Core i7 CPU, 3.40 GHz processor, 16 GB RAM computer). The FLOW-3D program, by using the same equipment and 8 cores, would require more than 70 days to solve the 72 hours flushing process of the complete reservoir. Hence, the three-dimensional simulations were only used to analyze the behavior of the flow in the first seconds of the flushing.

The results demonstrated the suitability of crossing different methodologies to achieve an adequate resolution of complex phenomena such as flushing operations. Thus, numerical simulations of differing degrees of complexity were used to complement the classical formulations and allow a better understanding of the physical phenomena.

Acknowledgments

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