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MINISTRY OF AGRICULTURE, IRRIGATION AND WATER DEVELOPMENT

SHIRE VALLEY IRRIGATION PROJECT

Hydraulic Modelling of Intake

Final numerical report

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1. INTRODUCTION

1.1. THE SHIRE VALLEY IRRIGATION PROJECT

Since the 1960s, Government of Malawi (GoM) has been interested in the implementation of Shire Valley Irrigation Project (SVIP) to develop irrigation in the Lower Shire Valley. Since then, the proposed project has been the subject of a large number of surveys and studies.

The GoM has requested financial assistance from the World Bank (WB) and the African Water Facility (AWF)/ African Development Bank (AfDB) for the preparation of the SVIP. Accordingly, the WB and AfDB are supporting the GoM with the preparation of comprehensive studies required to appraise the technical feasibility, economic/financial viability, environmental and social sustainability of the SVIP

Fig. 1. Shire River Basin

The SVIP is proposed to irrigate about 42,500 ha of land in the southern part of Malawi within the administrative districts of Chikwawa and Nsanje. The irrigable area is located on the west (right) bank of the Shire River in the Lower Shire River Valley (on the south of Shire River Basin, see Fig. 1). The intake of SVIP is proposed to be located upstream of Kapichira Dam[1].

1.2. PROJECT OBJECTIVES OF THIS STUDY

It is proposed to locate the SVIP intake upstream of Kapichira dam. As the introduction of an intake structure close to the Kapichira dam could have an adverse impact mainly on the reservoir

[1] Exact site to be determined by the present study.

sedimentation pattern and consequently on the concentration of sediment in the power plant intake and on the operation of the dam, it is of paramount importance to properly analyze the hydraulic behaviour around these structures under the various proposed configurations of the SVIP intake structure with various operational scenarios of the Kapichira dam. Furthermore the location of the SVIP intake could also have an impact on the suspended sediment concentration entering the SVIP main canal.

Consequently, it has been proposed by the Client, the AfDB and the WB to conduct a 3D mathematical modeling of the Shire river and Kapichira reservoir, including the existing power plant intake and the future SVIP intake to solve the problems related with the dynamics of sediment transport and river (reservoir area) morphology.

The objectives of the study are:

- to use the preliminary design of the intake general location and range of design discharges provided by the feasibility consultants, recommend the optimum site of the intake structure;
- to study the likely impacts of introducing the SVIP intake structure on the hydraulic behavior (incl. sedimentation) in the head-pond area and around the intake to iteratively determine the most optimal and efficient sediment exclusion and/or sediment ejection works to ensure safety and operational flexibility at the SVIP intake and Kapichira Power intakes. The scenarios considered in the iterative simulation exercise and the recommendations thereof should clearly demonstrate that the introduction of the SVIP intake structure and diversion of water to the feeder canal does not significantly affect the hydraulic performance of the two headworks;
- to study the reservoir sedimentation behaviour under various combination of the SVIP intake and Kapichira dam operations on the flushing regime and propose the appropriate modifications to ensure efficient flushing regime as determined by the hydraulic model simulation results; and
- to provide necessary guidance (feedback) based on the above objectives and findings as input to the Detail Design and other related studies of the SVIP.

To meet these requirements, various tasks have to be carried out during this study, mainly:

- hydrological and sedimentological data collection
- site visit
- meeting with the stakeholders
- meeting with other Consultants of the SVIP (mainly the Technical Feasibility consultancy)
- bathymetric campaign in the reservoir
- sedimentological campaigns (suspended sediment and bedload sediment)
- construction of 3D hydraulic and sedimentological numerical model of the reservoir and intakes
- tests of various intake locations and reservoir operations

1.3. PURPOSE OF THE REPORT

The purpose of this data Exploration Report is to present:

- the description of the numerical model,
- the results of the calibration of the model,
- a first assessment of different configurations of the proposed SVIP water intake.

2. SOFTWARE USED

2.1. THE TELEMAC SYSTEM

The computation software we will use is TELEMAC-3D, which is a part of the TELEMAC-MASCARET system. The TELEMAC-MASCARET system consists in a set of programs dedicated to environmental hydraulics problems: hydrodynamics, sedimentology, water quality, waves and subsurface flows. It is managed by a consortium that contributes to its development and dissemination and comprises Artelia, CEREMA and EDF in France, BAW in Germany, and Daresbury Laboratory and HR Wallingford in the UK.

ARTELIA, being a core member of the TELEMAC-ARTELIA consortium, has gained knowledge and experience on the software TELEMAC for more that twenty years, and has made numerous developments on it, especially for sediment transport.

2.2. TELEMAC 3D

TELEMAC-3D solves the three-dimensional hydraulics equations (non-hydrostatic Navier-Stokes equations in laminar or turbulent conditions), the transport-diffusion equations for tracers, using finite element or finite volume-type methods.

TELEMAC-3D also simulates the transport of suspended cohesive and non-cohesive sediment. To do so, it solves the suspended sediment transport and bed change equations. Exchanges with the bed are represented by the erosion and deposition flow terms.

The hydrodynamics can be recalculated at each time step on the basis of changes in the bed and density (internal coupling between flow and sedimentology).

2.3. DEVELOPMENTS BY ARTELIA FOR SEDIMENT TRANSPORT

The base version of TELEMAC 3D is limited at the present time to the transport of one class of sand and one class of cohesive sediment. ARTELIA has developed a version of TELEMAC 3D that exceeds the possibilities of the base version in terms of sediment transport.

In the case of sediment transport in the reservoir of Kapichira dam, we use a sediment transport model with the following specifications:

- sand sediment transport is computed with the total load formula (that takes into account both bedload and suspended transport) by Soulsby-Van Rijn (1997),
- clay is transported in suspension, the law of erosion used will be the one of Partheniades (1965), which is the most widely used for the erosion of cohesive sediment,
- silt is transported in suspension, also using the law of erosion by Partheniades.

A bed model has been developed in order to properly manage the evolution of the mass of the different sediment classes in the bed, and to take into account the composition of the sediment bed to compute erosion fluxes. An important physical process that is taken into account in this bed model is the sliding (collapsing) of the sediment slopes when they become steeper than a given critical angle of repose.

The specific parameters of the sediment transport model for Kapichira reservoir are presented in paragraph 5.1.

3. NUMERICAL MODEL OF KAPICHIRA RESERVOIR

3.1. MESH

The mesh used for the numerical computations consists in about 30000 mesh nodes. Mesh size ranges from 2 meters (near the dam), to 12 meters in the upstream part of the reservoir. Average mesh size in the reservoir is 8 meters.

This mesh is used to represent the reservoir in its present state (without SVIP intake). The mesh will be modified when taking into account this new intake.

3.2. BATHYMETRY

Two Digital Terrain Models have been built, that represent respectively the bathymetry of the reservoir just after the building of the dam (2001), and the present bathymetry, see Data Exploration Report (Chapter 3) for more details on the data used and their treatment. The bathymetry of the numerical model is directly based on these Digital Terrains Models (see below).

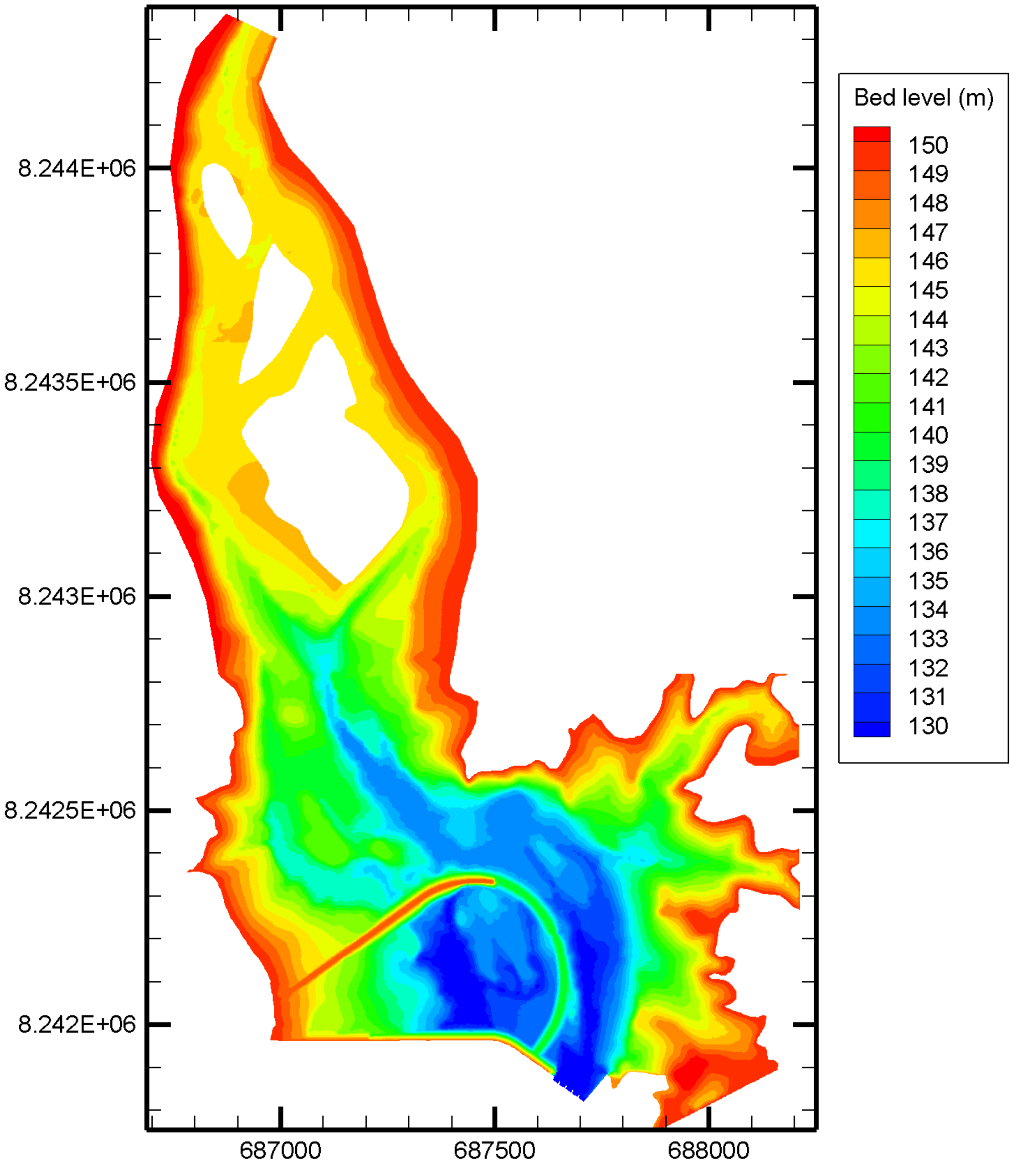

Fig. 2. Digital Terrain Model – 2001 Bathymetry

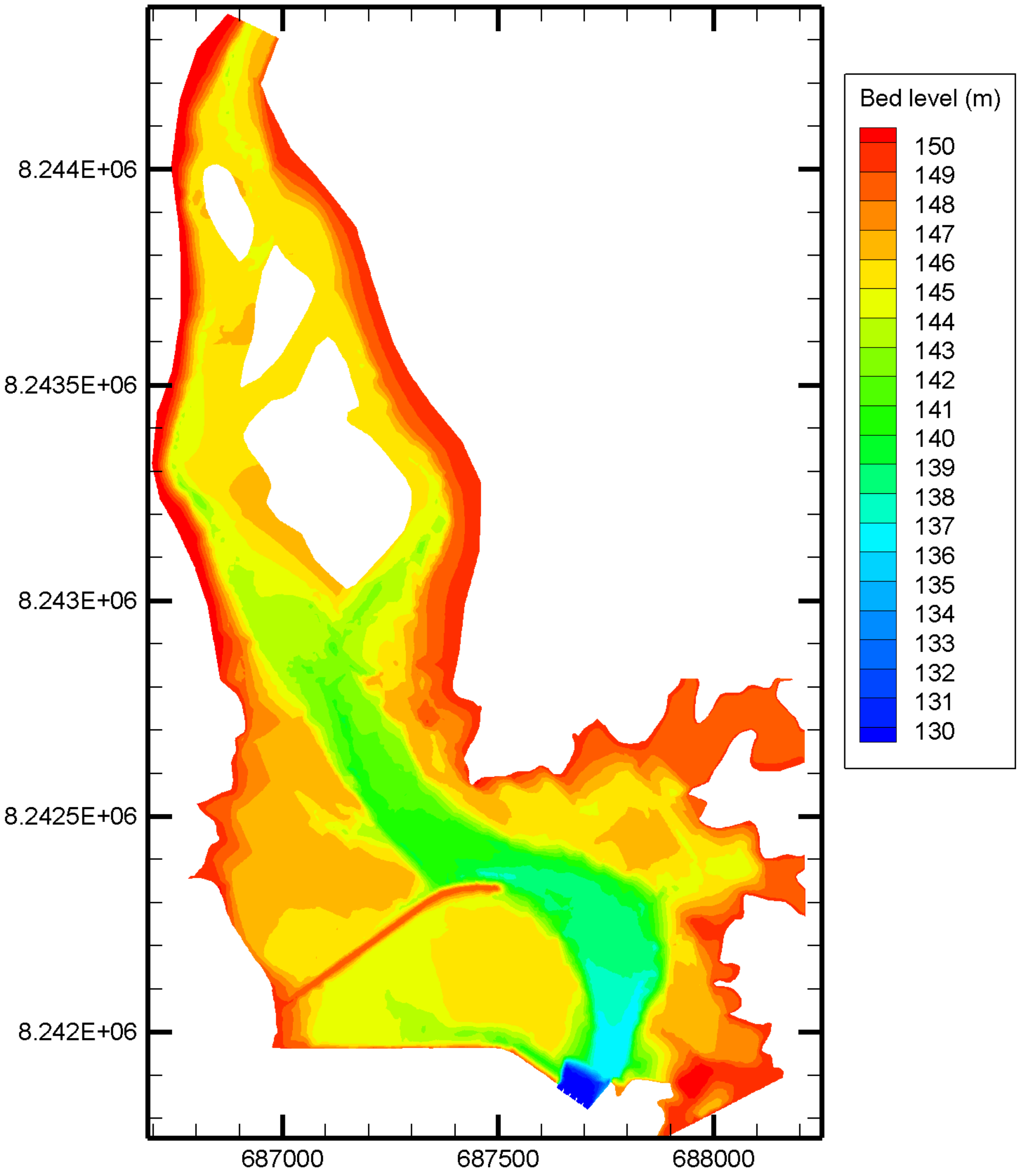

Fig. 3. Digital Terrain Model – 2016 Bathymetry

4. HYDRODYNAMIC COMPUTATIONS

4.1. PARAMETERS

The main parameter concerning the computation of the flow field in TELEMAC 3D is the coefficient of the friction law. The Manning-Strickler law is used. The Strickler coefficient K_s is linked to the manning coefficient n by $n=1/K_s$.

The Strickler coefficient used is of 30 $m^{1/3}/s$ for the areas with no sediment (and are thus probably rocky), and 45 $m^{1/3}/s$ for the areas covered with sediment, which are hydraulically smoother.

4.2. BOUNDARY CONDITIONS

There are seven liquid boundaries in the model of the present state:

- the upstream boundary
- the intake of the powerplant
- the five spillways

The boundary condition for all these liquid boundaries consists in an imposed discharge, either positive (incoming flow upstream) or negative (outgoing flow at the dam).

The implementation of the SVIP water intake in the model adds an eight boundary, with a negative imposed discharge.

4.3. HYDRODYNAMIC RESULTS - NORMAL FUNCTIONING OF THE DAM

The figure below presents an example of flow field in the reservoir during the normal functioning of the dam, both for the 2001 and the 2016 bathymetry. The hydrologic conditions are:

- water level in the reservoir 147 m,
- upstream discharge: 400 m^3/s
- discharge through the powerplant: 256 m^3/s
- discharge thourgh the spillways: 144 m^3/s (only first and last spillways are open)

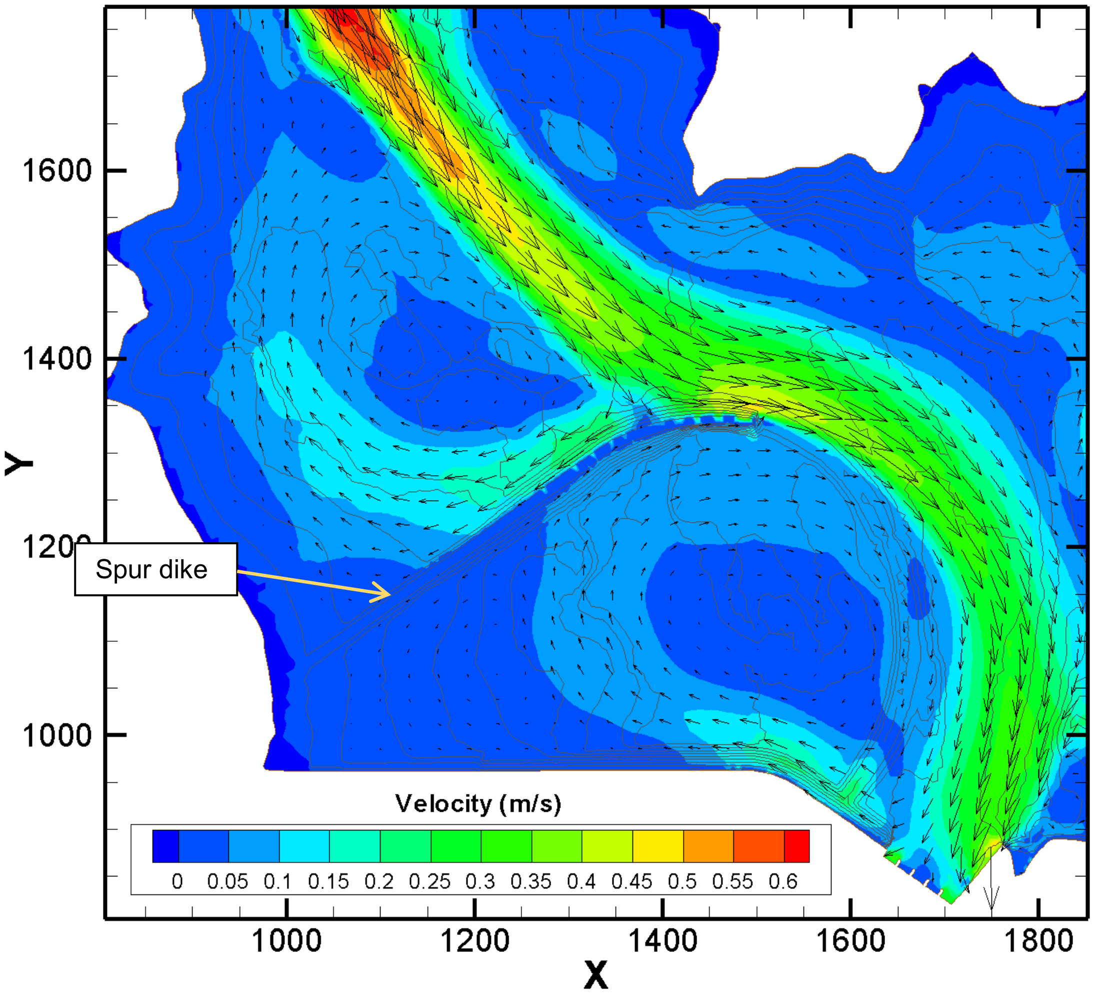

Fig. 4. Flow field– 2001 Bathymetry – 400 m³/s

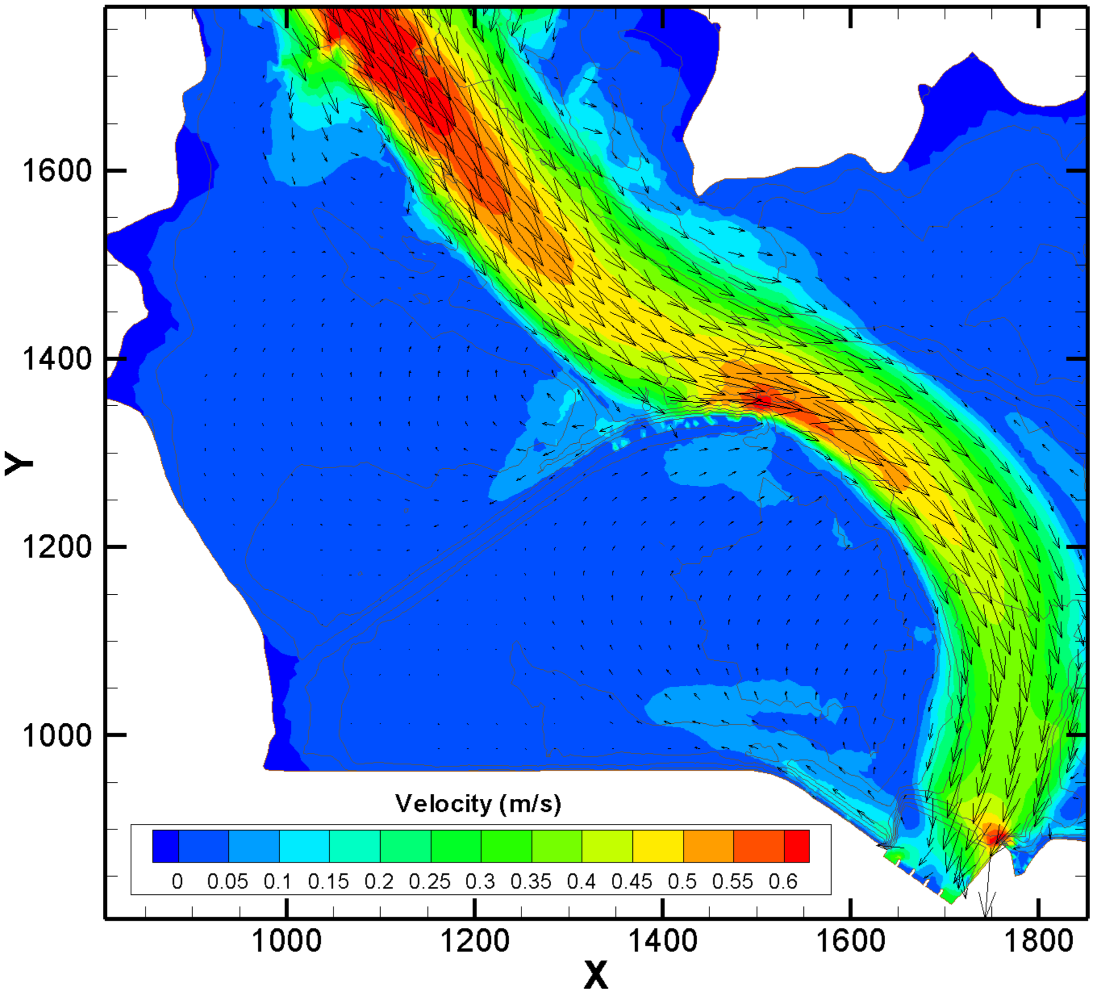

Fig. 5. Flow field– 2016 Bathymetry – 400 m³/s

This figure indicates the flow pattern in the reservoir: flow is concentrated in the main channel. There are two main recirculating cells: one upstream of the spur dyke, and the other downstream of the spur dyke. Flow velocity in these cells is very low in the present case (2016 bathymetry). It was higher (around ten centimetres per second) in the initial state of the reservoir.

4.4. HYDRODYNAMIC RESULTS - FLUSHING

The figure below presents an example of flow field in the reservoir during a flushing event, for the 2016 bathymetry

Discharge and water level vary fast during the flushing. The figure below shows an example of flow field during the flushing.

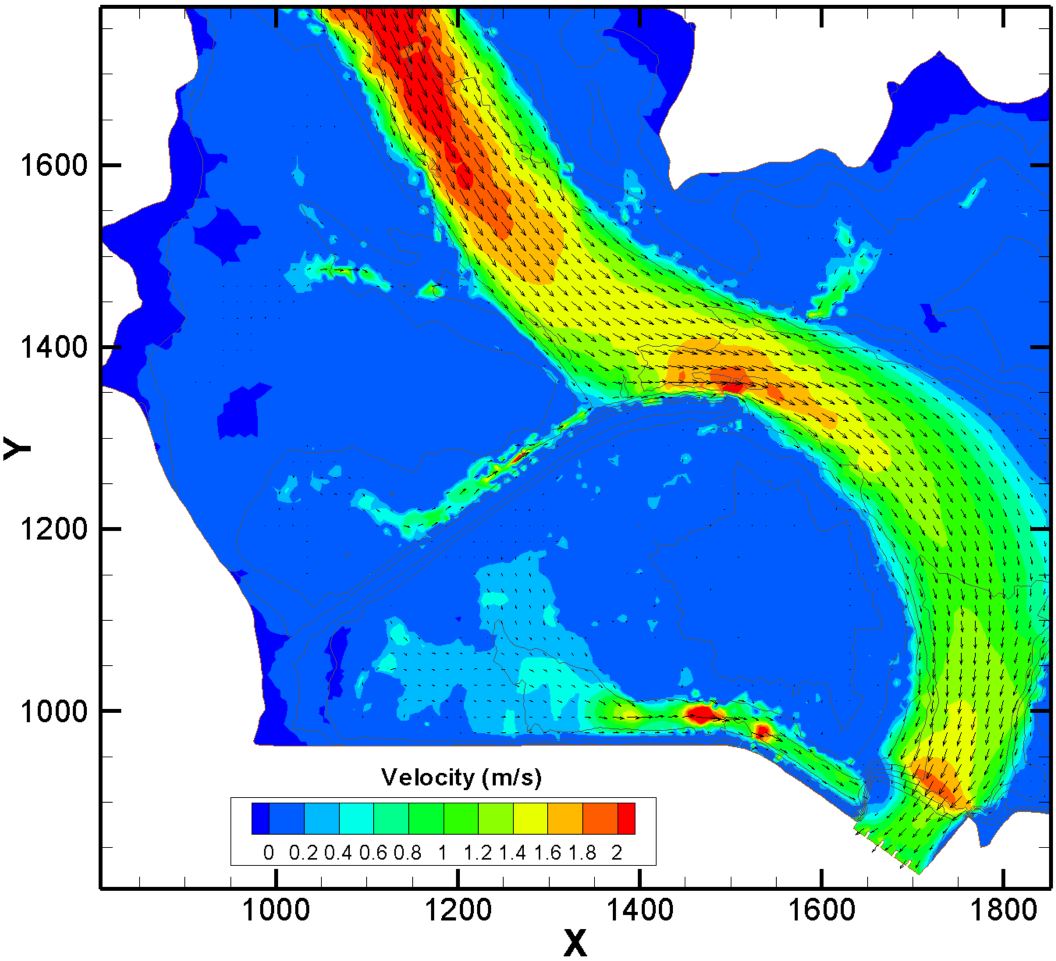

Fig. 6. Flow field– 2016 Bathymetry – Flushing – one hour after the start of the flushing

The flow during flushing is very strong, with velocities of up to 2 m/s in the main channel.

5. SETUP OF THE SEDIMENT TRANSPORT COMPUTATIONS

5.1. PARAMETERS

Three classes of sediment are considered, to account for sand, silt and clay.

The grain size of the sand class is set to 0.14 mm, according to the results of the survey presented in Chapter 5 of the Data Exploration Report.

The grain size of the fine sediment classes (silt and clay) are not a parameter of the transport model. Their fall velocity is used instead. Based on the fall velocity measurements presented in the paragraph 5.4 of the Data Exploration report, the fall velocity of the silt class is set to 1.2 mm/s while the fall velocity of the clay class is set to 0.175 mm/s. This corresponds to grain sizes of about 40 microns and 15 microns respectively.

The critical angle of repose of the sediment in the bed is set to 9°, based on the bed slopes of the sediment deposits observed in the 2016 bathymetry.

5.2. BOUNDARY CONDITIONS

Sediment load must be imposed at the upstream boundary of the model.

This sediment load is computed, as an approximation, as a function of the river discharge. Indeed, high sediment concentrations in suspension tend to occur during high river discharge, while low concentrations occur at low discharge. There are other factors (for instance the time of the year, or if a flood occurs shortly after another one) in play in the concentration of sediment in suspension, but very long and detailed times series of suspension measurements would be necessary to be able to take them into account.

Using the data presented in Chapter 4 of the Data Exploration Report, the sediment in suspension at the upstream boundary of the numerical model is defined as a function of the river discharge using the relation presented in the figure below:

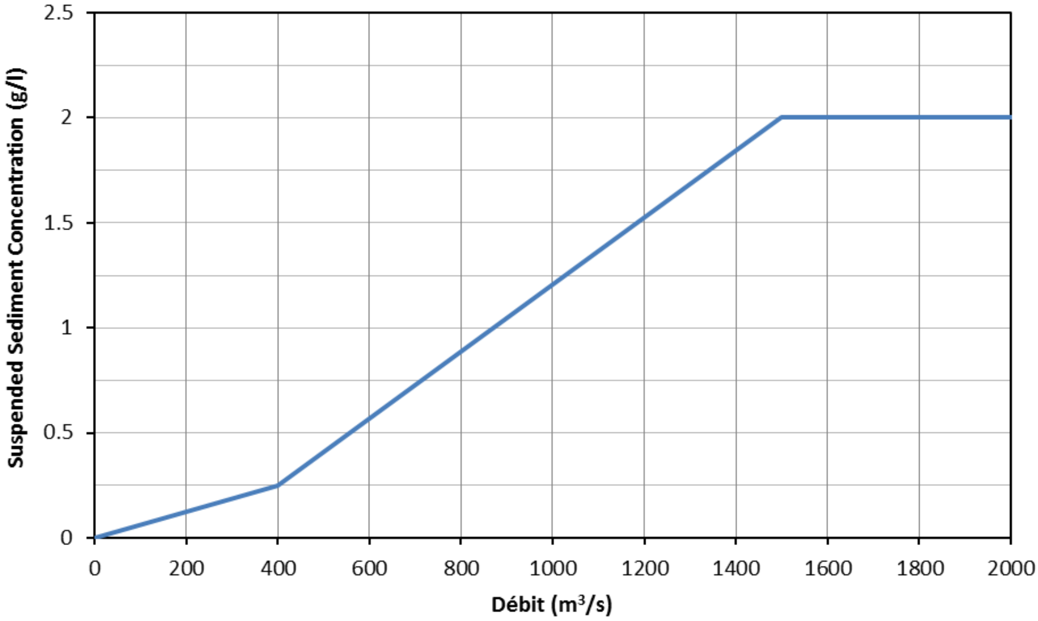

Fig. 7. Relation between river discharge and sediment concentration at the upstream boundary

Of this concentration, 20% is silt while 80% is clay.

At the downstream boundaries of the model (intake of the power plant and spillways), sediment exits the model freely. In order to ensure stability of the model during the run, the bed immediately upstream the spillways (on a length of 10 meters) is automatically and continuously cleared of sediments. In reality, this area is probably cleaned mostly during flushings.

6. CALIBRATION OF THE SEDIMENT TRANSPORT MODEL

The objective of this stage of calibration is to simulate the morphological evolution (mainly deposition) in the reservoir since the building of the dam in 2001.

6.1. BOUNDARY CONDITIONS

The time-series of incoming discharge in the reservoir (from 2001 to 2016) have been reconstructed (see paragraph 2.3 of Data exploration Report for details).

The hypotheses about management of discharge at the dams are the following:

- Environmental flow (minimum discharge through the spillways) : 17 m^3/s
- Discharge through the turbines : 128 m^3/s for 2001-2013, 256 m^3/s for 20014-2015;

The water level in the reservoir is regulated at a fixed value of 146.5 m.

The flushings during this period will be taken into account according to the data we have on the flushings of the period from 19/03/2015 to 27/12/2015 (described by spillway discharges and water level in the reservoir at a 30 minutes time-step). Based on this, we will use the following hypothesis for the flushings:

- Frequency: every two months
- duration: eight hours
- water level in the reservoir during the flushing : 142.9 m

ESCOM does probably not perform flushing during low flows, but as we had no information on their criterion for triggering the flushing, the constant time step of two months was retained.

The resulting discharges at the different boundaries of the model for the calibration period are presented in the figure below.

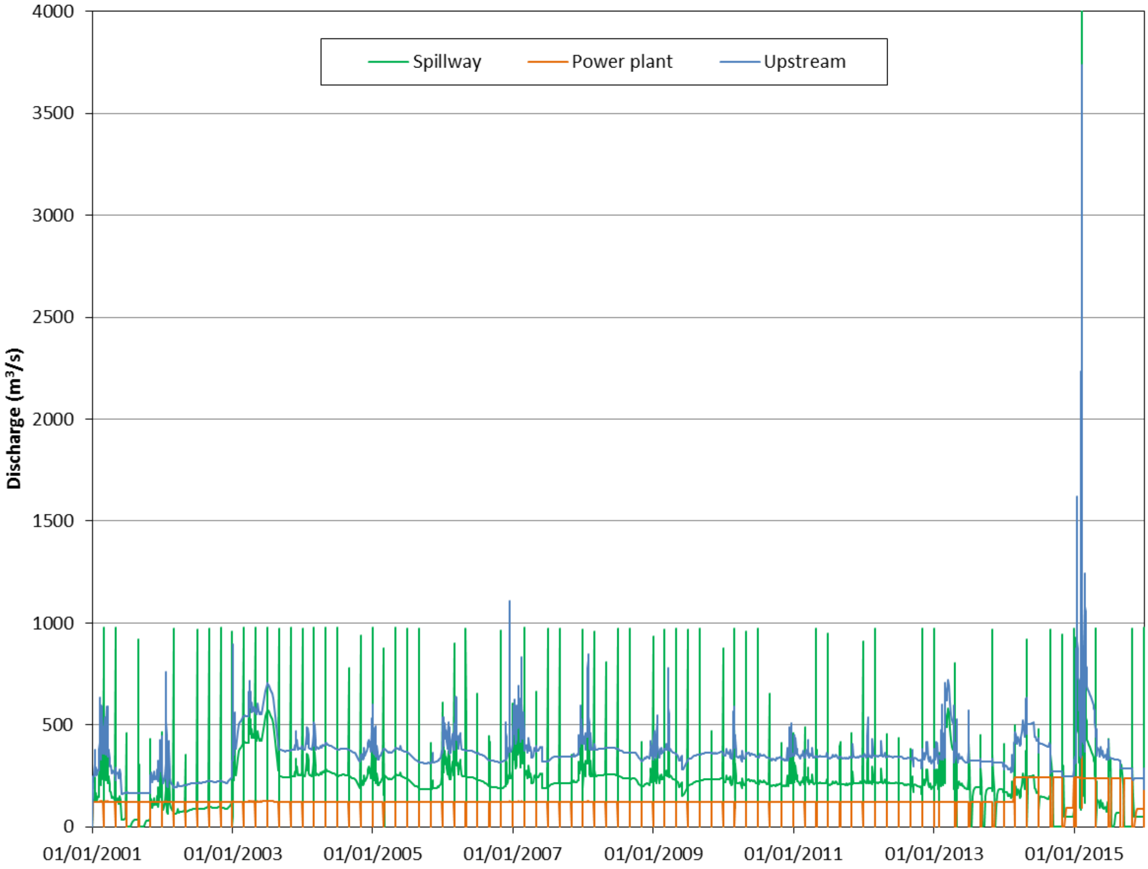

Fig. 8. Liquid discharges during the calibration period

The regular peaks of spillway discharge at about 1000 m^3/s correspond to the flushings.

The sediment load imposed at the upstream boundary of the numerical model is computed according to the hypothesis presented in paragraph 5.2. Using this with the time-series of incoming discharge in the reservoir, the cumulated mass of fine sediment entering the reservoir is computed and presented in the figure below.

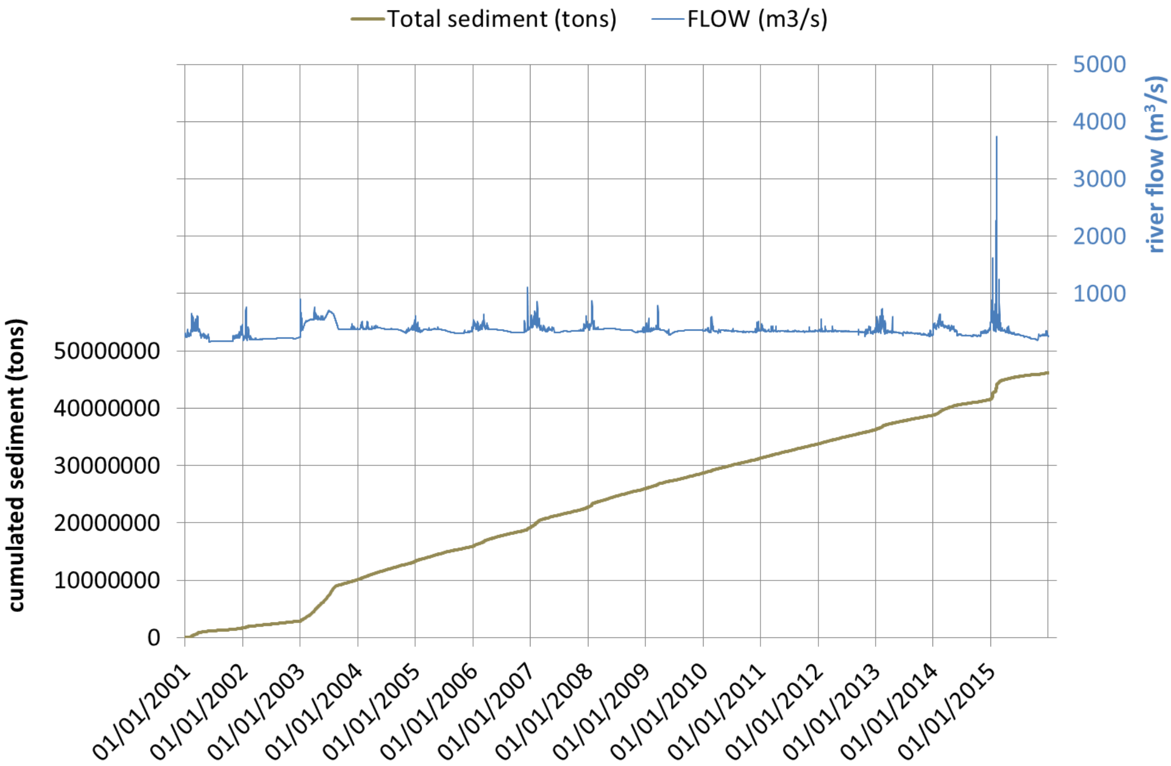

Fig. 9. Cumulated mass of fine sediment entering the reservoir in the numerical model

According to our hypothesis, about 46 millions of tons of sediment (consisting in sand, silt and clay) have entered the reservoir between 2001 and 2016.

6.2. CALIBRATION PROCESS

The duration of a full computation run is of two days on 48 cores of ARTELIA's parallel computing server.

A large number of calibration computations have had been to be run in order to tune the inputs and parameters that are not fully determined by the results of the sediment survey. These parameters include:

- Choice of the friction coefficient for the hydraulic model
- Amount of sand entering the model
- Amount of fine sediment (silt and clay) entering the model (remaining in acceptable range according to the survey)
- Parameters (rate of erosion and critical erosion shear stress) of the erosion law for fine sediment
- Diffusion coefficient for the sediment in suspension

In total, dozens of computations have been run before reaching satisfactory results for the calibration. The results of the optimal run are presented below.

6.3. RESULTS

The comparison between the measured 2016 bathymetry and the computed 2016 bathymetry is presented on the figure below.

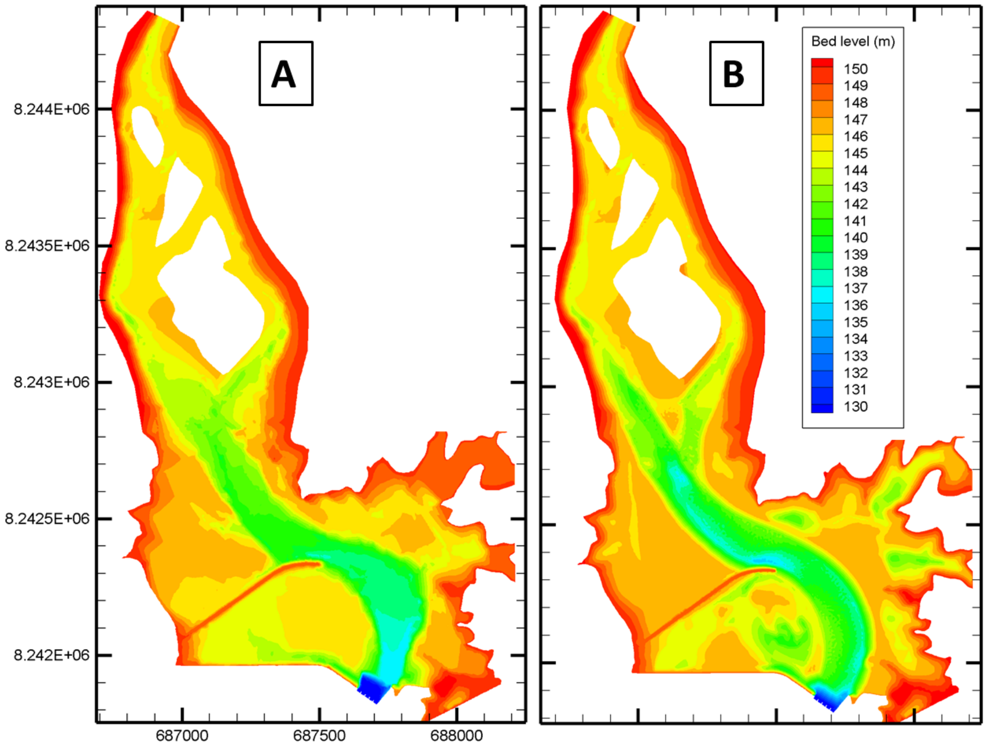

Fig. 10. Comparison between 2001 bathymetry and 2016 bathymetry – A: measured, B: computed

The figure below presents the comparison between the measured bed evolution (equal to the thickness of sediment deposit) and the computed bed evolution between 2001 and 2016.

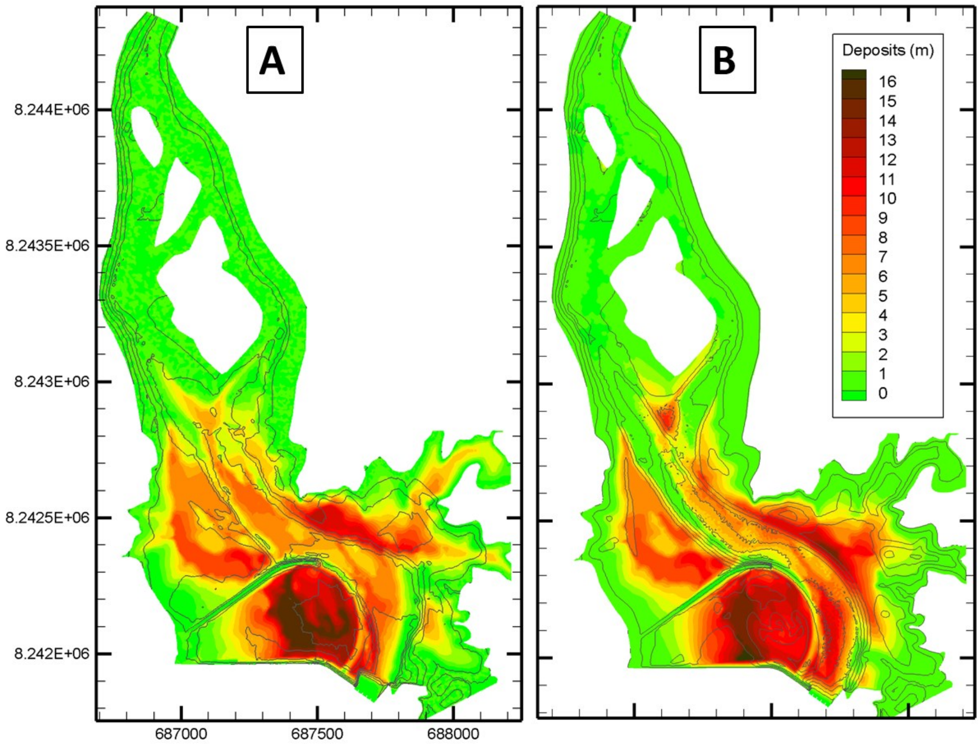

Fig. 11. Comparison between measured (A) and computed (B) bed evolution

The evolution of the volume of the sediment deposited in the reservoir during the run is presented in the figure below.

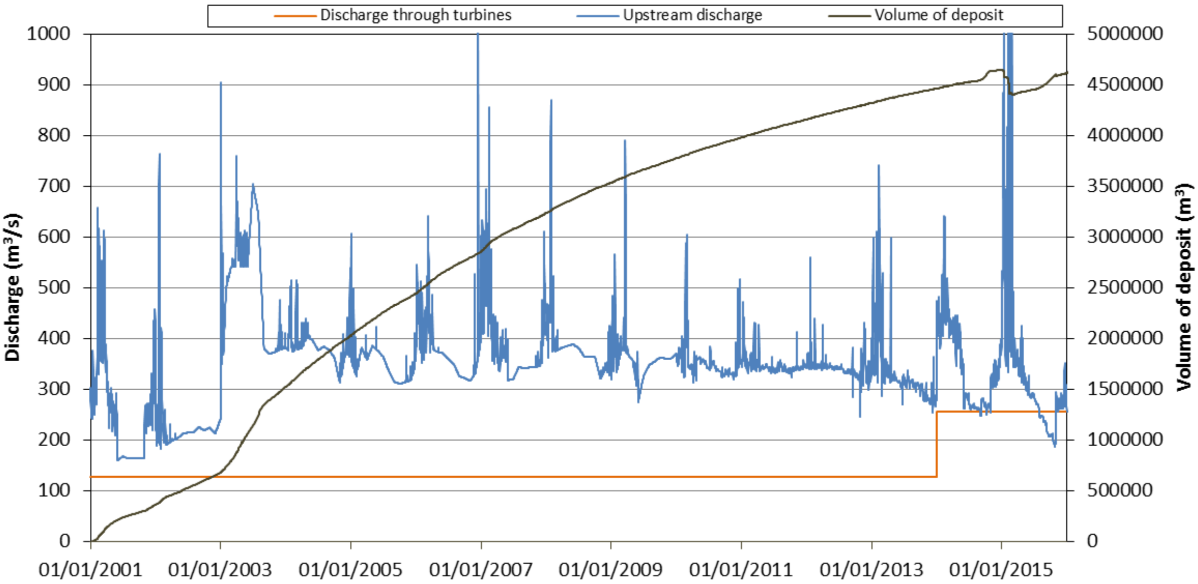

Fig. 12. Evolution of the volume of sediment in the reservoir

6.4. ANALYSIS

The general pattern of deposition is well simulated; the model is able to create the two large sediment deposits laying on the right side of the reservoir. The characteristics of the main channel flowing through the reservoir are also well reproduced, both in width and depth.

The volume of deposits on the whole reservoir is correct, as are the volume of deposits in the right side deposit upstream of the spur dyke and right side deposit downstream of the spur dyke. The comparison between the measured and computed volumes is presented in the table below.

Tabl. 1 - Volume of deposit between 2001 and 2016 – comparison between model and measurements

Zone	Measured volume (m^3)	Computed volume (m^3)
Upstream RHS deposit	724 000	804 000
Downstream RHS deposit	1 562 000	1 499 000
Entire reservoir	4 628 000	4 663 000

The time evolution of the volume of sediment in the reservoir (see Fig. 12) enables to precise what are the role of flood, low discharge periods, and flushings in the sedimentary functioning of the reservoir.

At the beginning of the run, when the reservoir is empty or nearly empty of sediment, one can see that, except during flushings, all hydrologic conditions lead to an increase of the volume of sediment deposits in the dam. In particular, floods bring a large increase of sediment.

At the end of the run, when there are large sediment deposits, floods tend rather to expel sediment from the reservoir; Low floods still bring an increase of sediment deposits, but at a slower rate than at the beginning of the run. This indicates the reservoir is close to having reached full capacity. Looking at the 2016 bathymetry, it is clear that the only place where significant deposit can still take place is on the downstream right hand side deposit.

7. ASSESSMENT OF DIFFERENT INTAKE CONFIGURATIONS

7.1. METHODOLOGY

7.1.1. Boundary conditions

The hydrologic scenario that is used as a model input in order to test the different configurations of the water intake is the period of two years from 2014 (see figure below).

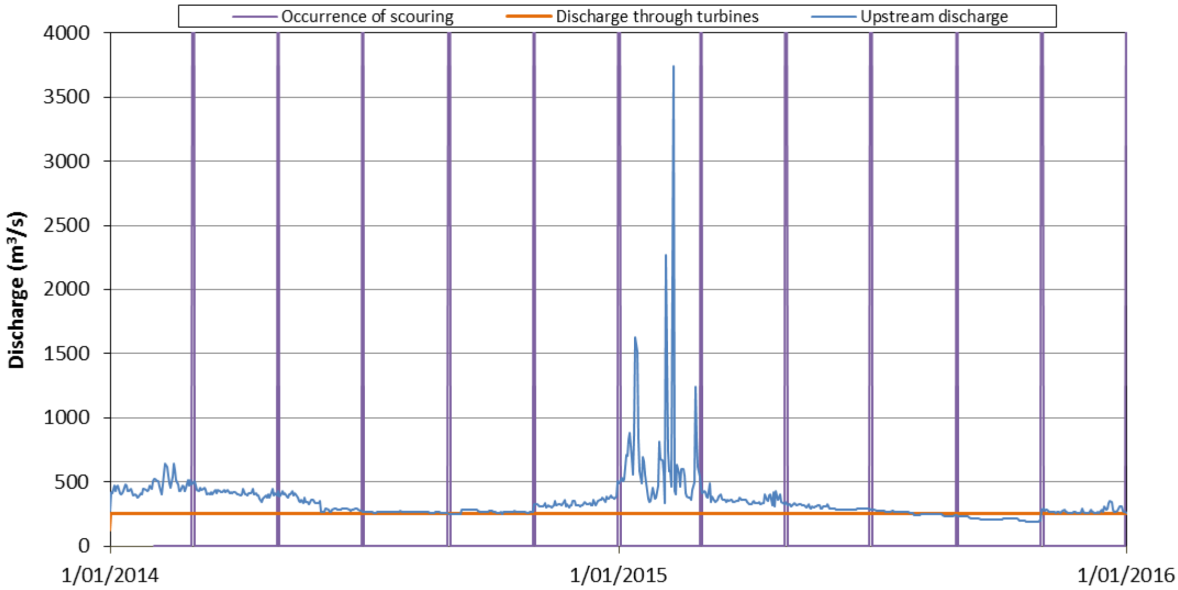

Fig. 13. Liquid discharges during the assessment period

This period enables to test the influence of all kind of hydrologic regimes for the Shire river, from low flows to very high discharges.

The hypothesis used for dam management and flushings are the same than for the calibration run (see 6.1). The only difference is that the target water level in the reservoir during normal functioning is now 147 m rather than 146.5 m (according to ESCOM this target level was raised recently).

The discharge through the turbines remains of 256 m^3/s and the environmental flow remains of 17 m^3/s.

The water discharge diverted from the reservoir by the new intake is 50 m^3/s. There is no discharge at the water intake during flushings.

During this two-year period, the amount of sediment entering the model is 5.74 million tons of clay, 1.44 million tons of silt, and 181 000 tons of sand.

Two durations for the hydrologic scenarios are considered. First, the two-years hydrological sequence presented above is used in order to compare all four tested configurations

7.1.2. Overview of tested configurations

In this preliminary report, four configurations of the SVIP water intake are tested in the numerical model. These configurations were decided in agreement with the Technical Feasibility Consultant

The plans of structure of the water intake have been provided by the Technical Feasibility Consultant.

- Configuration 1: the intake is located just downstream of the spur dyke.
- Configuration 2: the intake is located just downstream of the spur dyke, and the whole right hand side deposit downstream of the spur dyke is dredged to a level of 141 m. The volume of dredging is 560 000 m^3.
- Configuration 3: the intake is located just upstream of the spur dyke.
- Configuration 4: the intake is located just upstream of the spur dyke, and the whole right hand side deposit upstream of the spur dyke is dredged to a level of 141 m. The volume of dredging is 640 000 m^3.

The figures below show the different configurations.

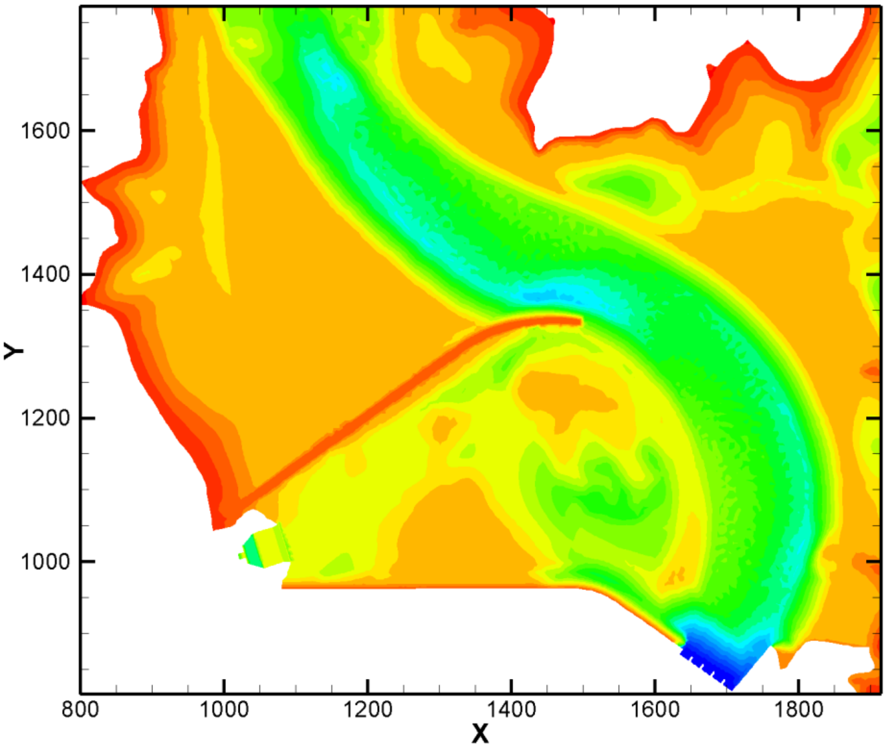

Fig. 14. View of configuration 1

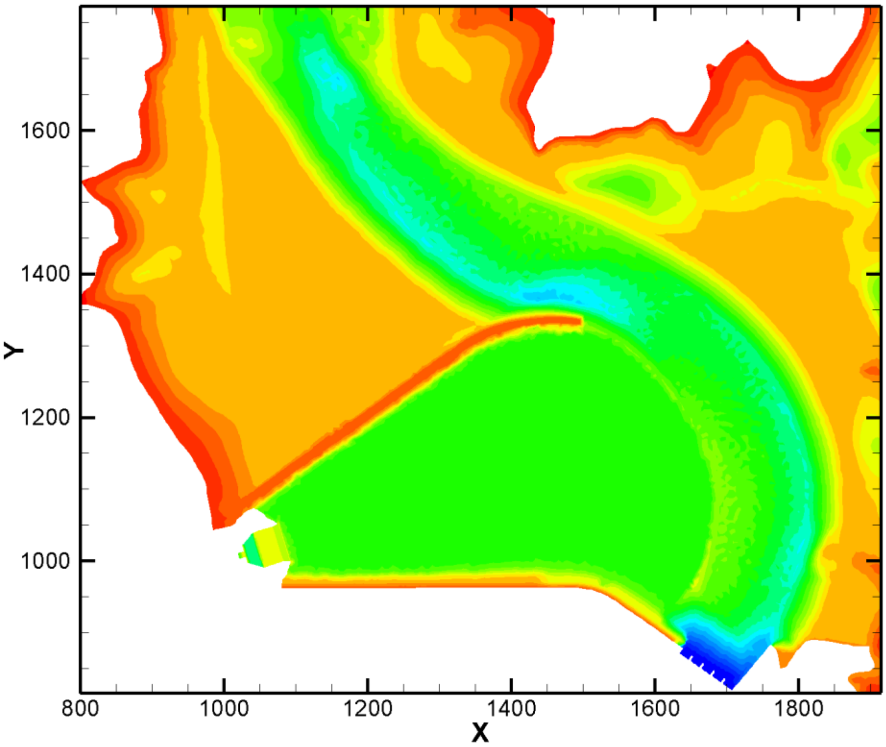

Fig. 15. View of configuration 2

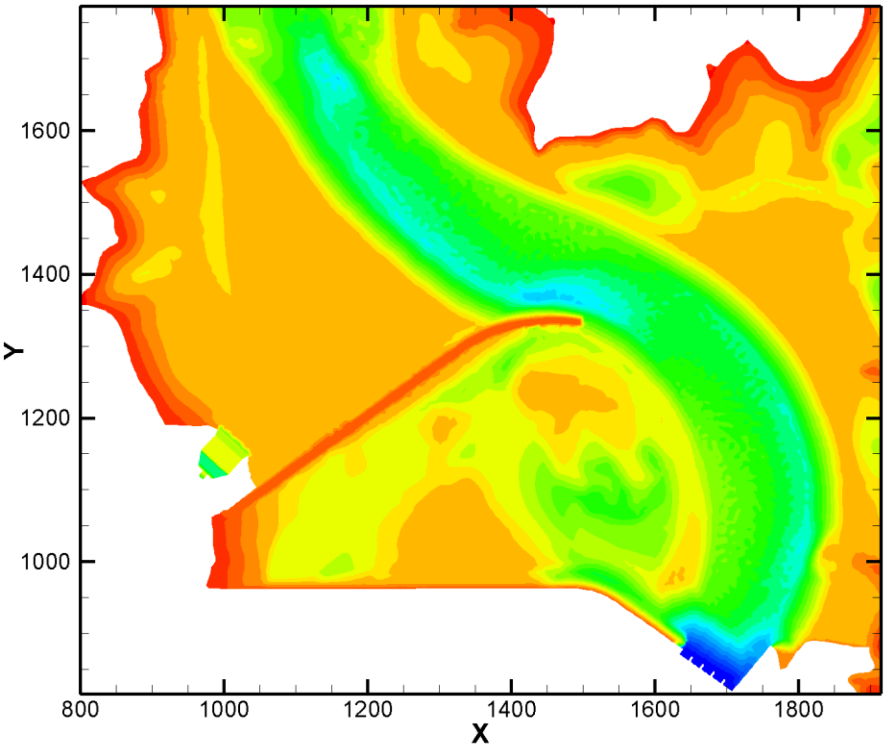

Fig. 16. View of configuration 3

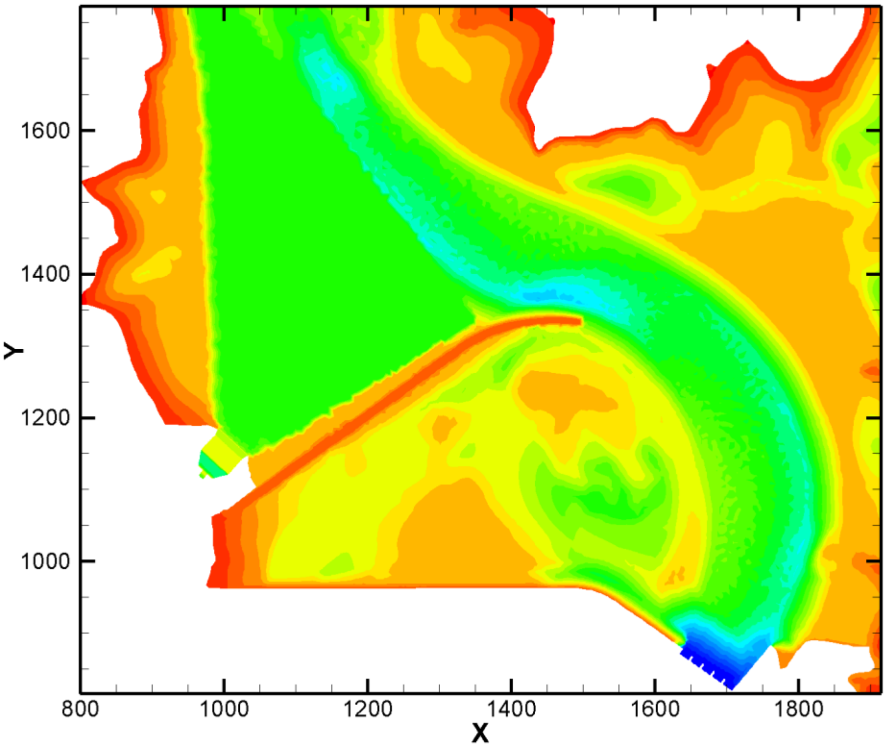

Fig. 17. View of configuration 4

The bathymetry of the reservoir (outside the intake and the eventual dredging) consists in the computed bathymetry at the end of the calibration run. This enables to initialize properly the composition of the sediment mixture (fractions of sand, silt and clay) in the bed at the start of the assessment run using the computed composition at the end of the calibration run.

7.2. RESULTS

First, it appeared that with configuration 3, the intake is hydraulically not able to take 50 m^3/s. It was thus not possible to test this configuration with the specified hypothesis.

The figures below show the cumulated mass of sediment transiting through the SVIP intake during the run, for the three considered configurations (1, 2 and 4). Note that in all three cases sand does not reach the intake during the run.

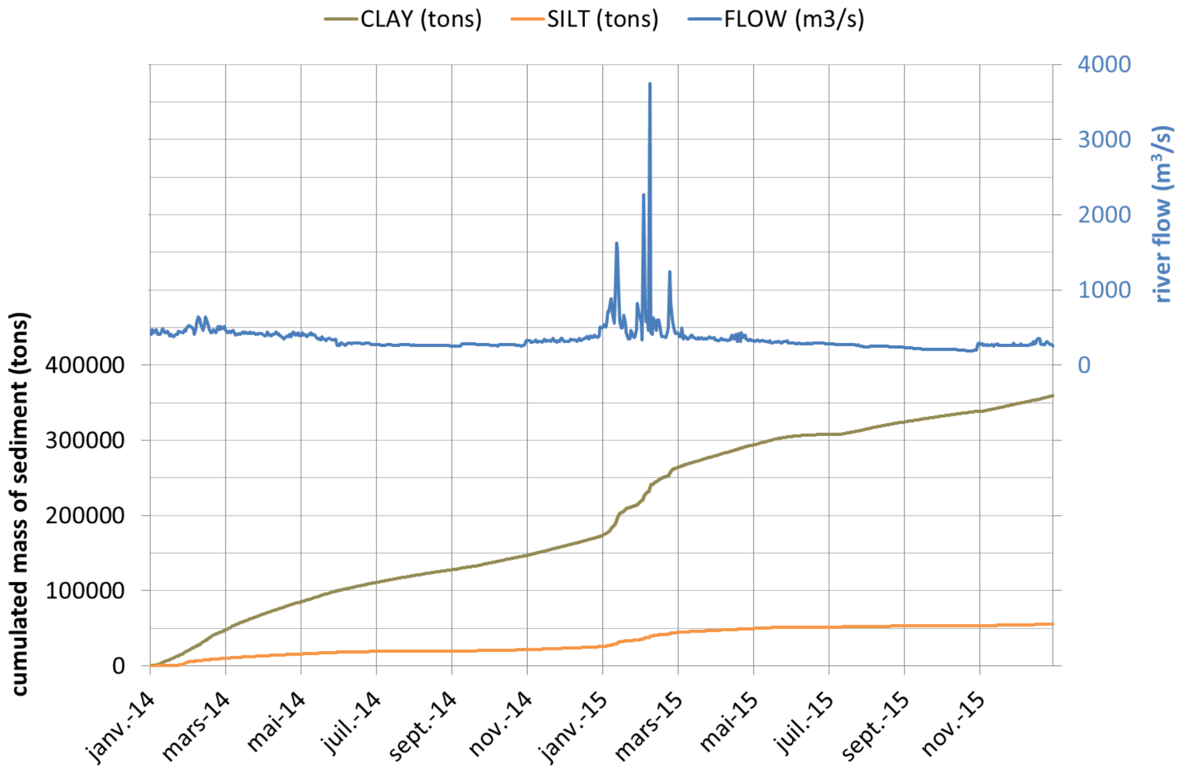

Fig. 18. Mass of sediment entering the intake – configuration 1

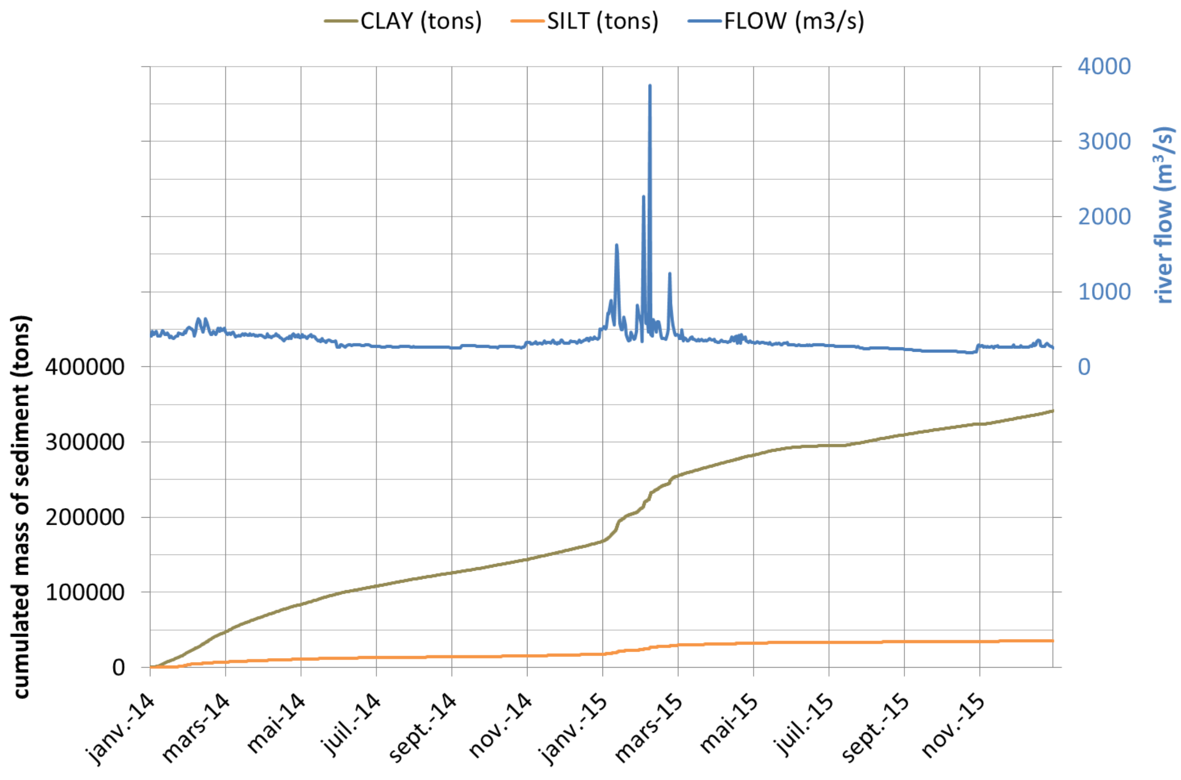

Fig. 19. Mass of sediment entering the intake – configuration 2

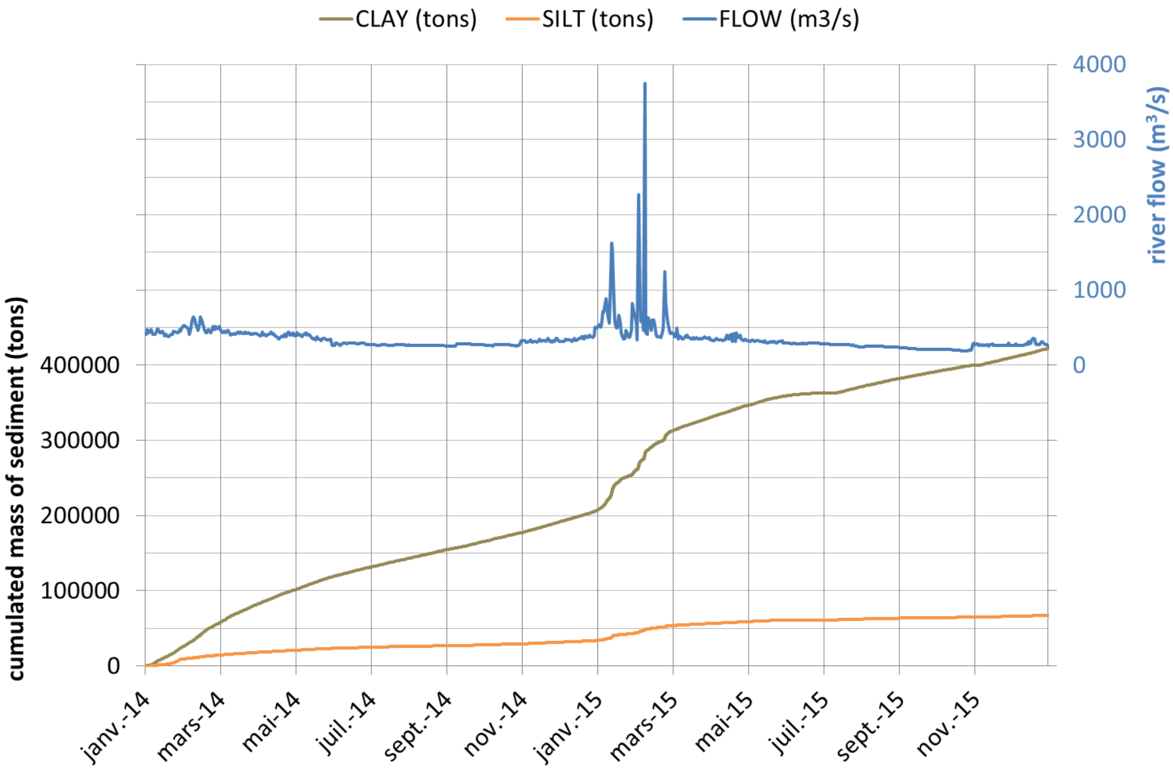

Fig. 20. Mass of sediment entering the intake – configuration 4

The table below list the final mass of sediment having transited during the computations

Tabl. 2 - Mass of sediment entering the intake during the two-year run

Configuration	Silt (tons)	Clay (tons)	Total (tons)
1	55 500	357 900	413 400
2	35 900	340 300	376 200
4	67 300	421 800	489 100

The evolutions of the volume of sediment deposits during the runs have been computed for the three configurations. Three areas are considered: the upstream right-hand-side deposit, the downstream right-hand-side deposit, and the whole reservoir. The results are presented below.

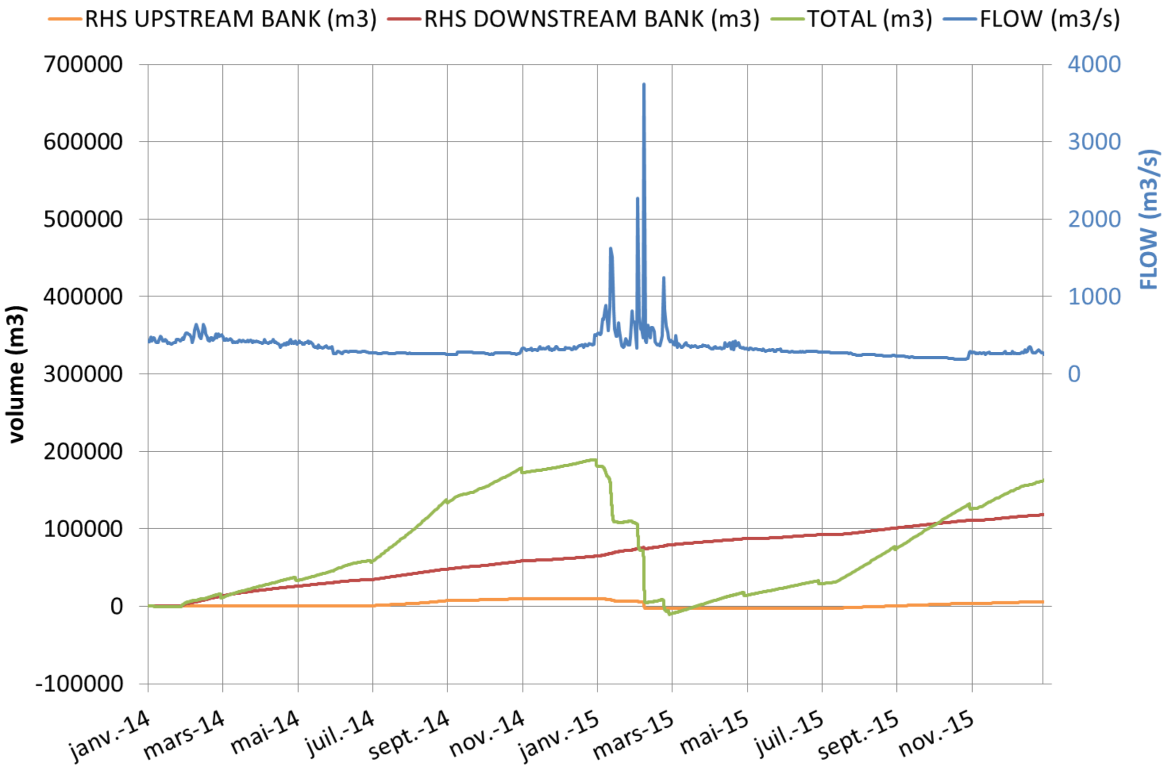

Fig. 21. Evolution of sediment deposits – configuration 1

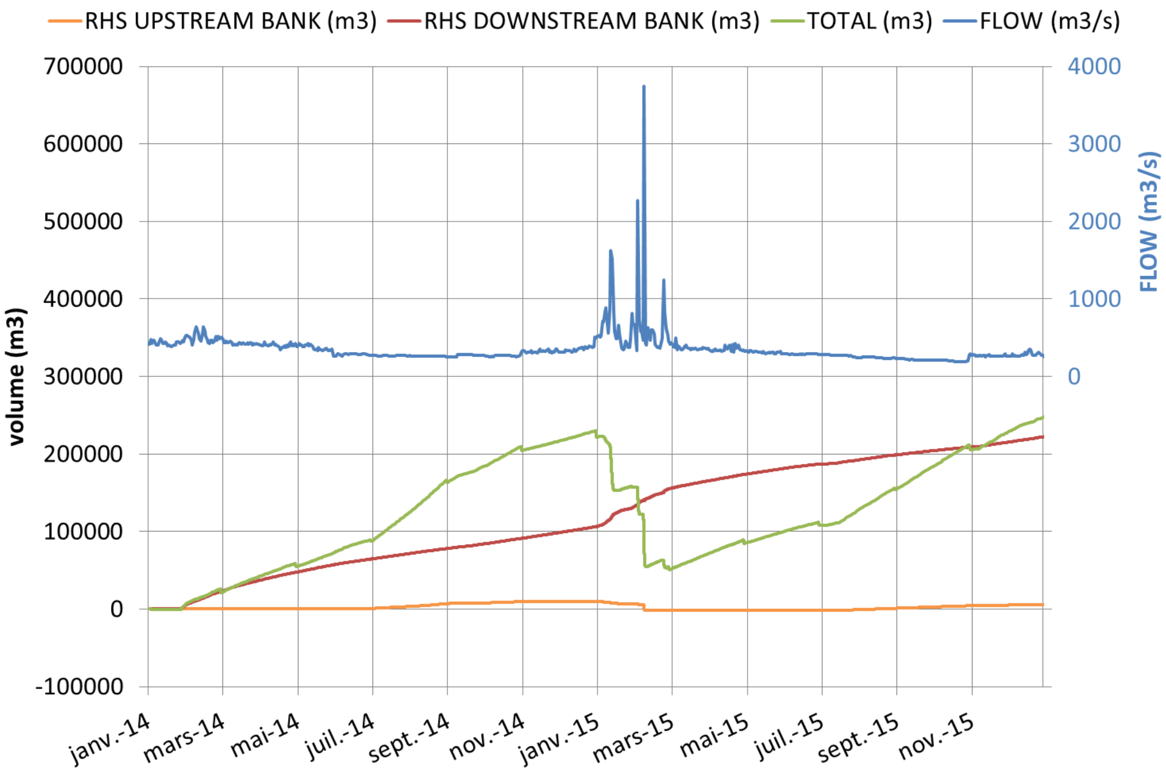

Fig. 22. Evolution of sediment deposits – configuration 2

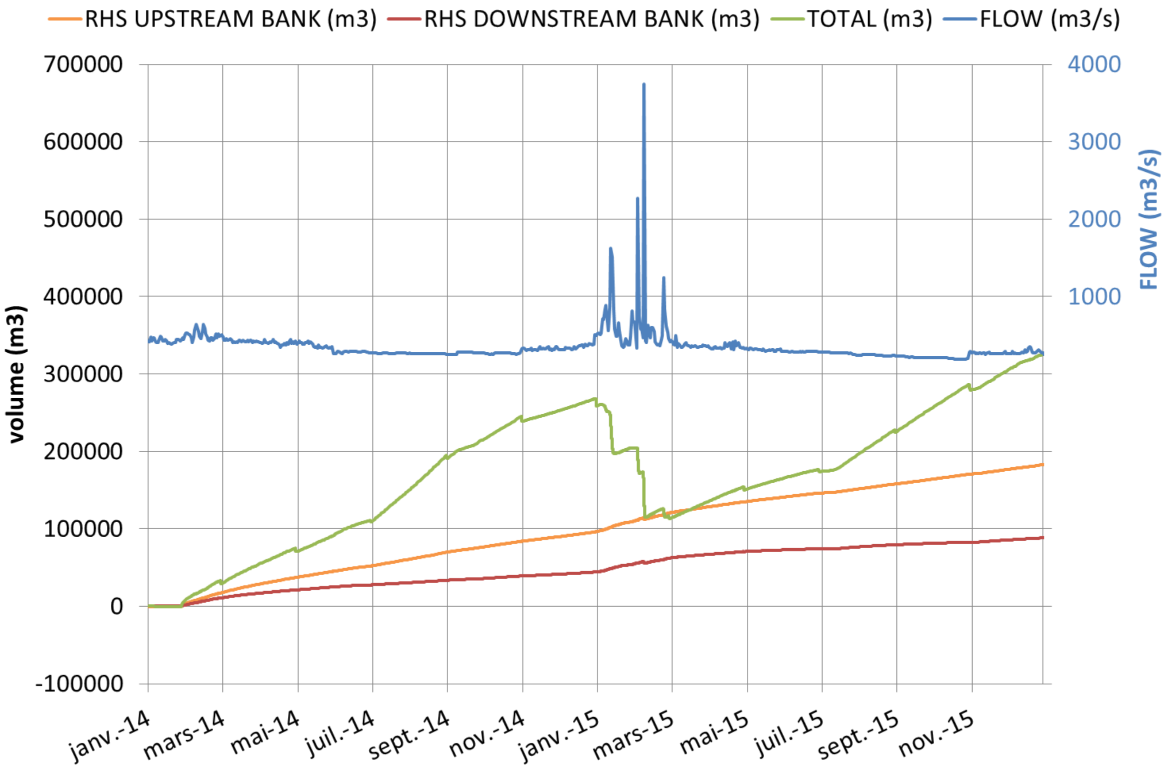

Fig. 23. Evolution of sediment deposits – configuration 4

The table below lists the final results in terms of evolution of volume of sediment in the reservoir for the three runs.

Tabl. 3 - Evolution of the volume of sediment in the reservoir during the two-year run

Configuration	**RHS upstream** deposit (m^3)	**RHS downstream** deposit (m^3)	**Whole reservoir** (m^3)
1	5 300	118 100	163 300
2	5 700	222 400	248 400
4	183 200	88 400	327 300

7.3. ANALYSIS

Logically, period of floods tend to bring more sediment in the intake than period of low flows. During low flow, there is still a significant amount of clay entering the intake, but few silt.

For all configurations, the mass of sediment entering the intake is quite large, between 190 000 and 250 000 tons per year. About 90% of this sediment consists in very fine sediment.

However the comparison between the different configurations tested show that the downstream location of the intake (as in configurations 1 and 2) is more beneficial than the upstream location as it enables to have significantly less sediment entering the intake.

Dredging the downstream sediment deposit enables to reduce the amount of sediment entering the intake, but the gain obtained by the dredging is not huge.

The runs enable to show how the reservoir responds to dredging. It appears that the dredged areas (for configurations 2 and 4) undergo significant deposition during the two-year computations. The upstream right-hand-side deposit (dredged in configuration 4), with 90 000 m^3 of deposits per year, traps sediment faster than the downstream right-hand-side deposit (dredged in configuration 2), which sees an added 50 000 m^3 of deposits per year compared to the non-dredged configuration 1.

8. LONG TERM IMPACT OF INTAKE AND ROLE OF DREDGING

8.1. METHODOLOGY

Now that the optimal location of the intake has been confirmed (see previous chapter), the model is used to determine:

- the long term bathymetric evolution of the reservoir with or without the SVIP intake,
- the impact of the SVIP intake on the power plant intake from an hydro-sedimentary point of view,
- the impact of eventual dredging on the hydrosedimentary functioning of the reservoir.

8.1.1. Boundary conditions

The hydrologic scenario that is used as a model input in order to test the different configurations of the water intake is five times the period of two years from 2014 (see figure below), with a maximum discharge set to 600 m^3/s, to avoid overrepresentation of the large floods during this scenario.

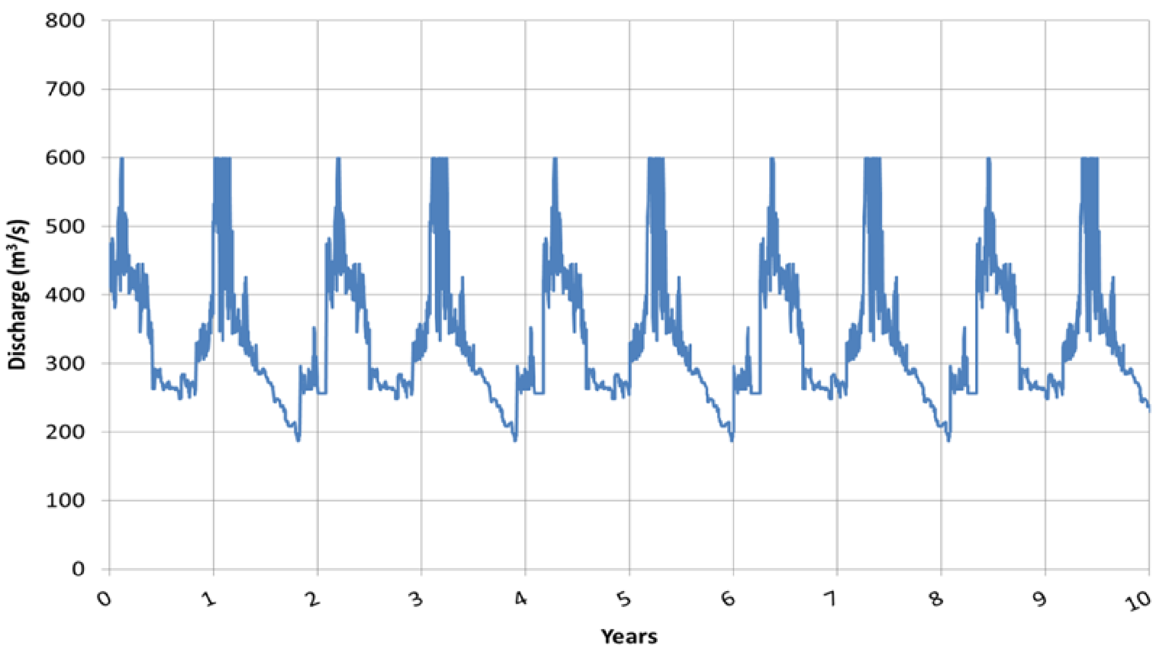

Fig. 24. Liquid discharge during the long-term period

The hypothesis used for dam management and flushings are the same than previously:

- Target water level in the reservoir during normal functioning: 147 m;
- Discharge through the turbines: 256 m^3/s;

- Environmental flow remains of 17 m^3/s;
- Water discharge diverted from the reservoir by the new intake: 50 m^3/s. There is no discharge at the water intake during flushings.

8.1.2. Configurations tested

The different configurations of the reservoir that are studied with these ten-years computations are:

- a reference state : "natural evolution of the reservoir, with no SVIP intake.
- five computations with the SVIP intake located downstream of the spur dyke :
  - two computations with no dredging in the reservoir, with two SVIP intake discharges: 25 and 50 m^3/s.
  - three different locations for dredging in the reservoir, with the SVIP intake discharge set to 50 m^3/s.

The table below recapitulates these different configurations.

Tabl. 4 - Listing of computations

Name	Intake	Initial bathymetry	Dredging	Intake discharge (m^3/s)
Ref	None	2016 – computed	None	0
A	Downstream	2016 – computed	None	25
B	Downstream	2016 - computed	None	50
C	Downstream	2016 - computed	Downstream RHS deposit	50
D	Downstream	2016 – computed	Upstream RHS deposit	50
E	Downstream	2016 – computed	LHS deposit	50

Downstream RHS deposit means the large right-hand-side deposit that lies downstream of the spur-dyke. Upstream RHS deposit means the large right-hand-side deposit that lies upstream of the spur-dyke. LHS deposit is the left-hand-side deposit that lies about 500 meters of the power plant intake.

The three dredged configurations are presented in the figures below.

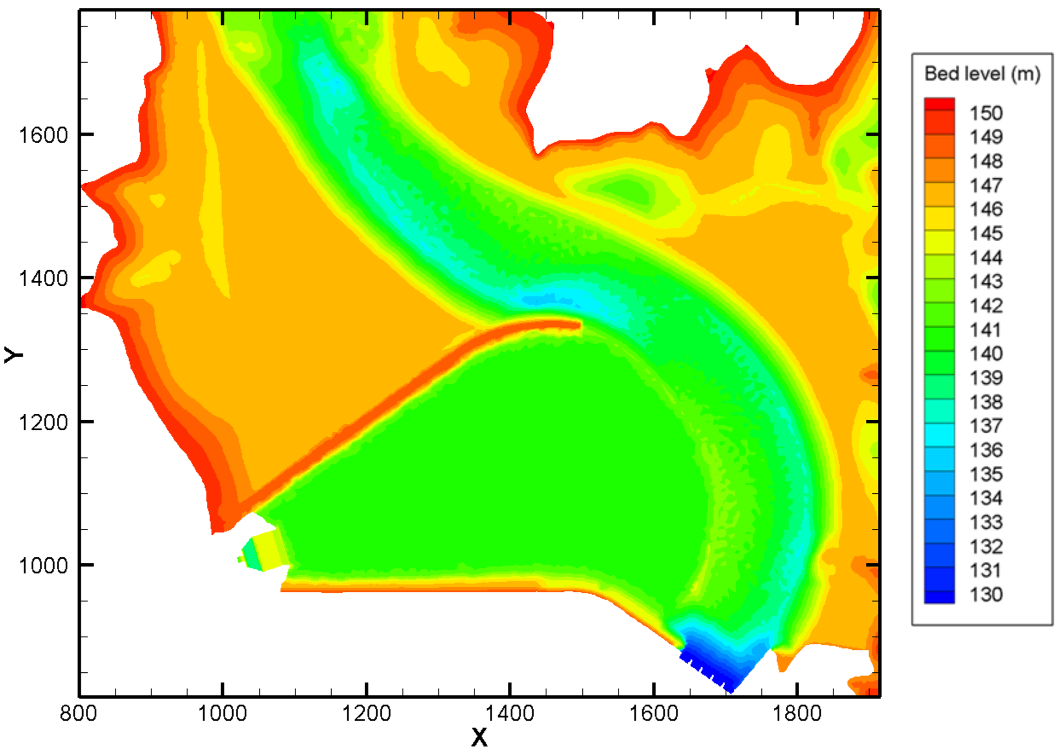

Fig. 25. View of configuration C with downstream RHS dredging

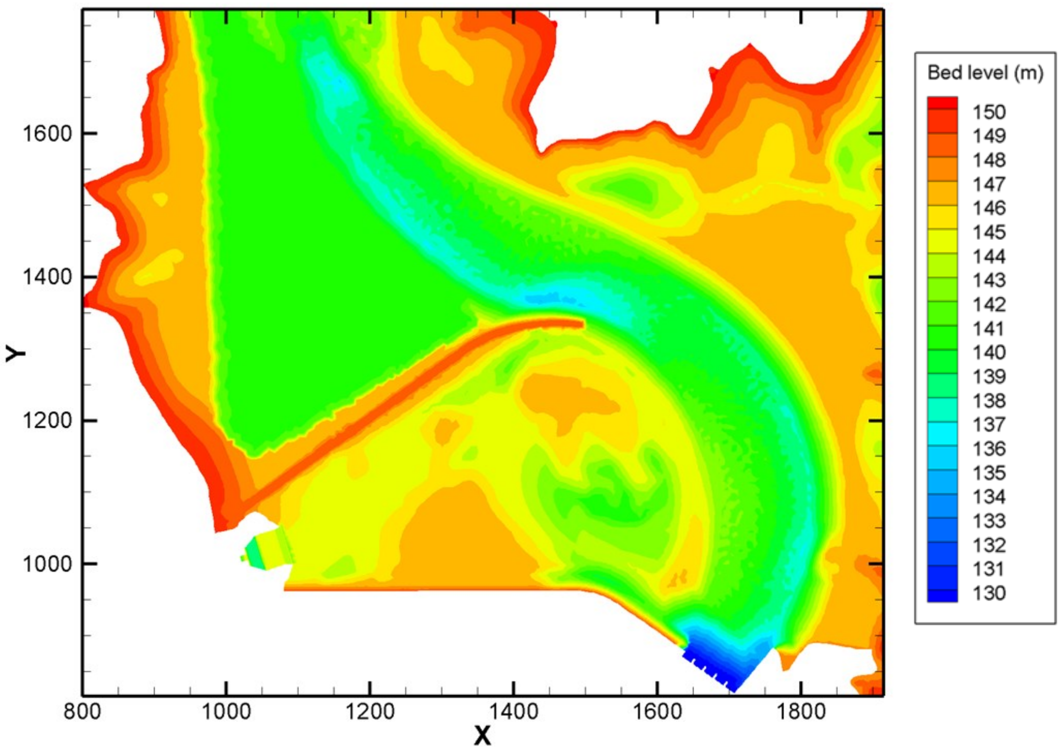

Fig. 26. View of configuration D with upstream RHS dredging

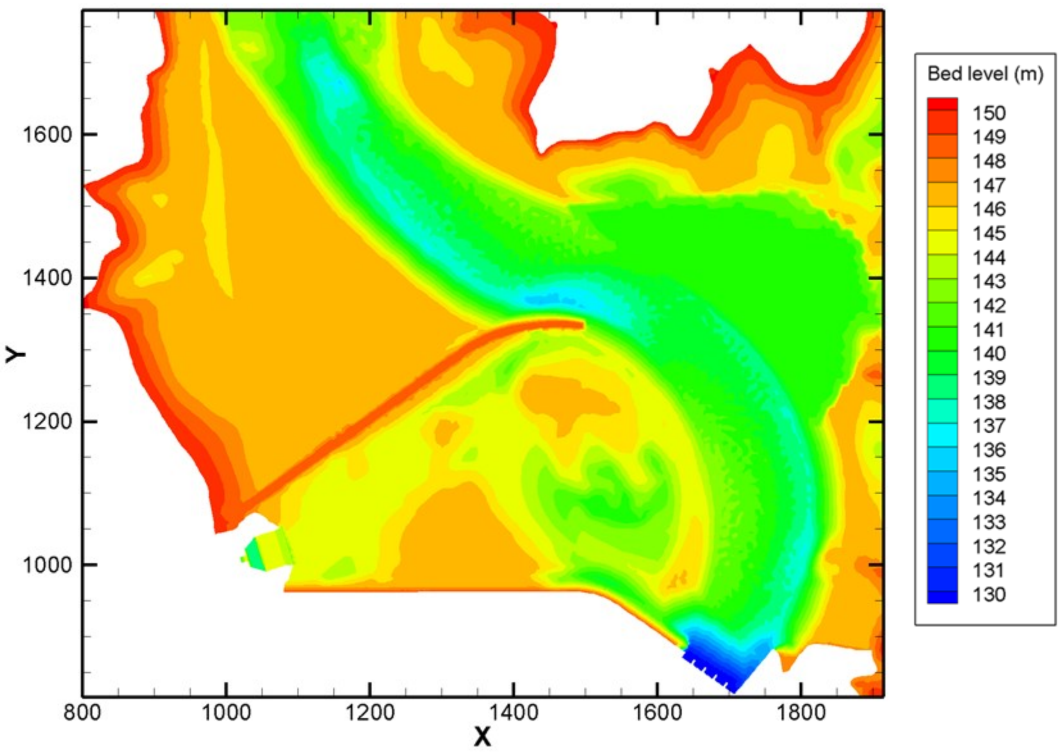

Fig. 27. View of configuration E with LHS dredging

The volumes dredged are presented in the table below.

Tabl. 5 - Volume dredged for the configurations involving dredging

Dredging location	Volume (m^3)
Downstream RHS	560 000
Upstream RHS	640 000
LHS	300 000

8.2. RESULTS

The final bathymetry computed for the six configurations are presented in the figures below.

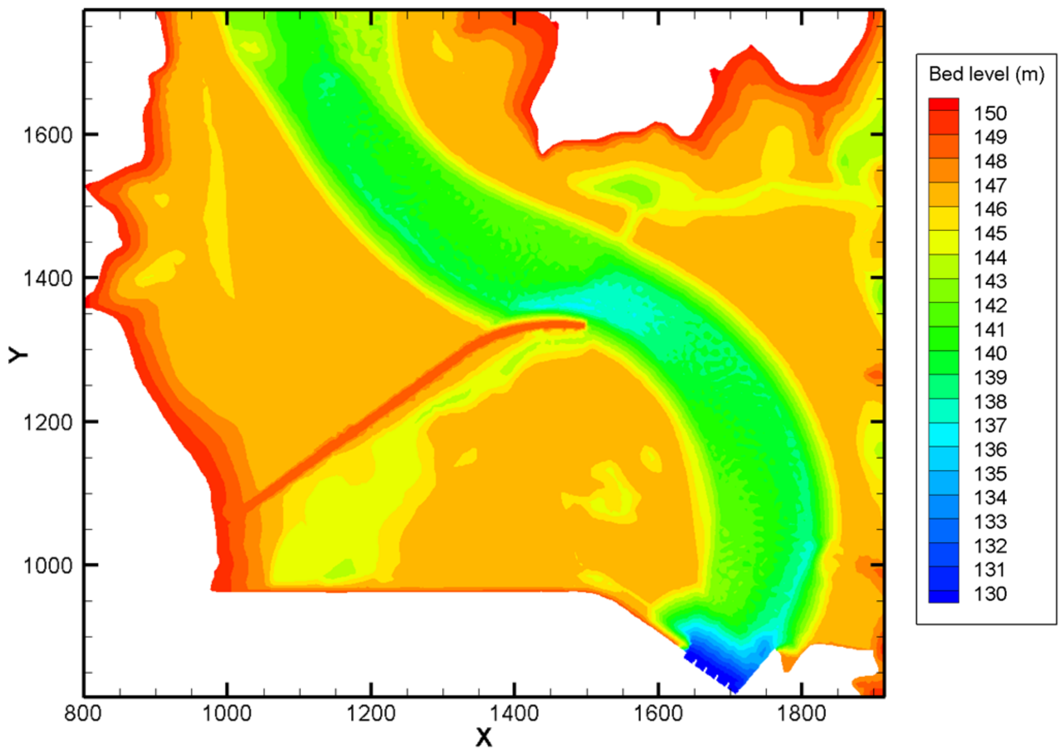

Fig. 28. Bathymetry after 10 years – Reference computation

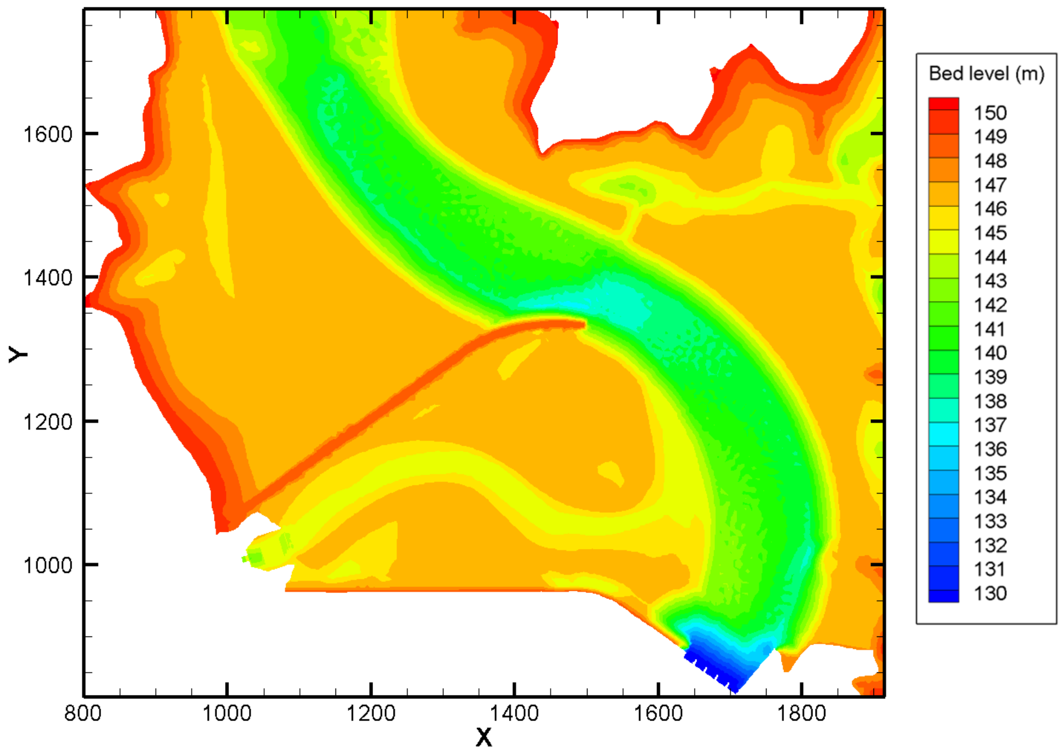

Fig. 29. Bathymetry after 10 years – Configuration A

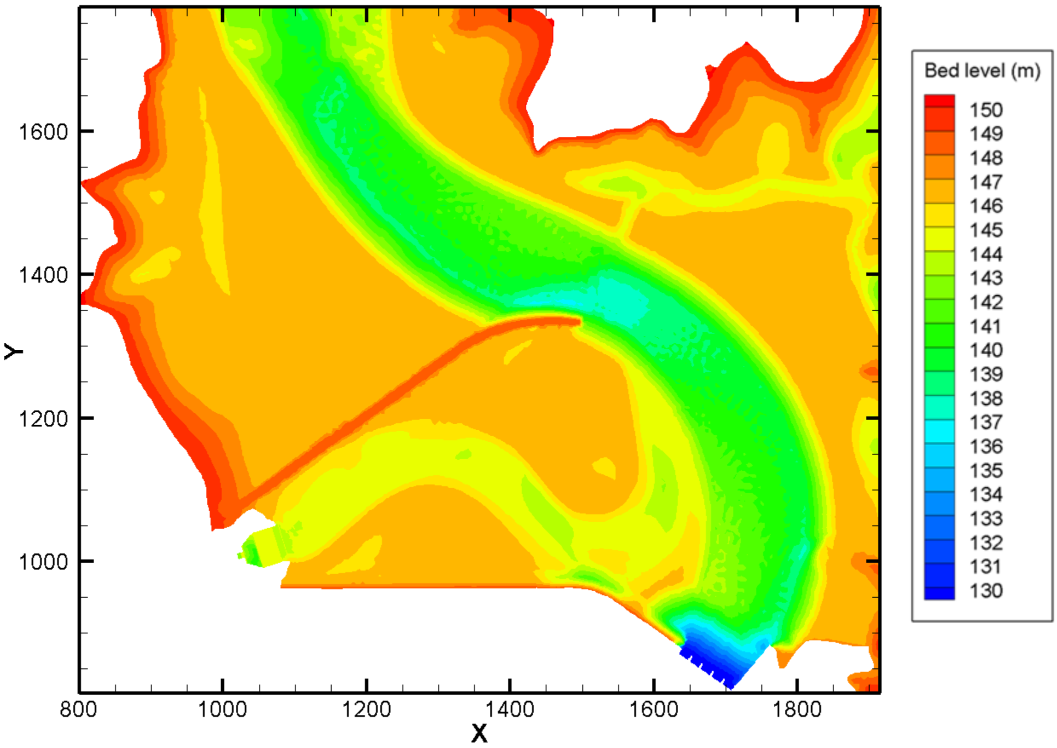

Fig. 30. ***Bathymetry after 10 years – Configuration B***

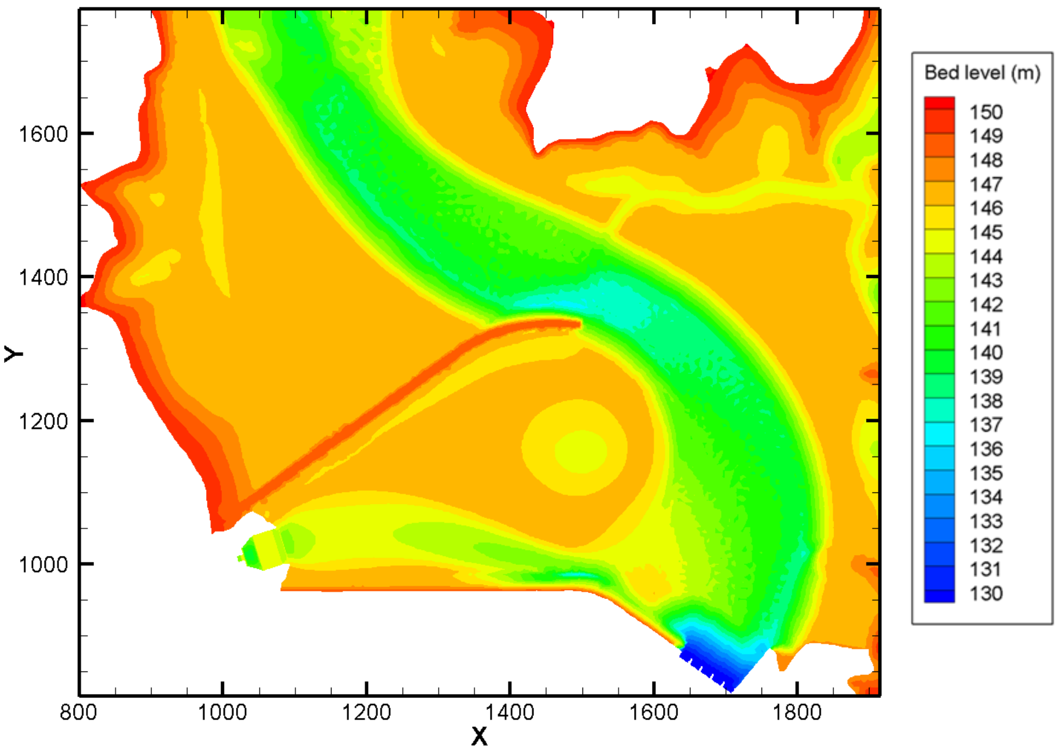

Fig. 31. ***Bathymetry after 10 years – Configuration C***

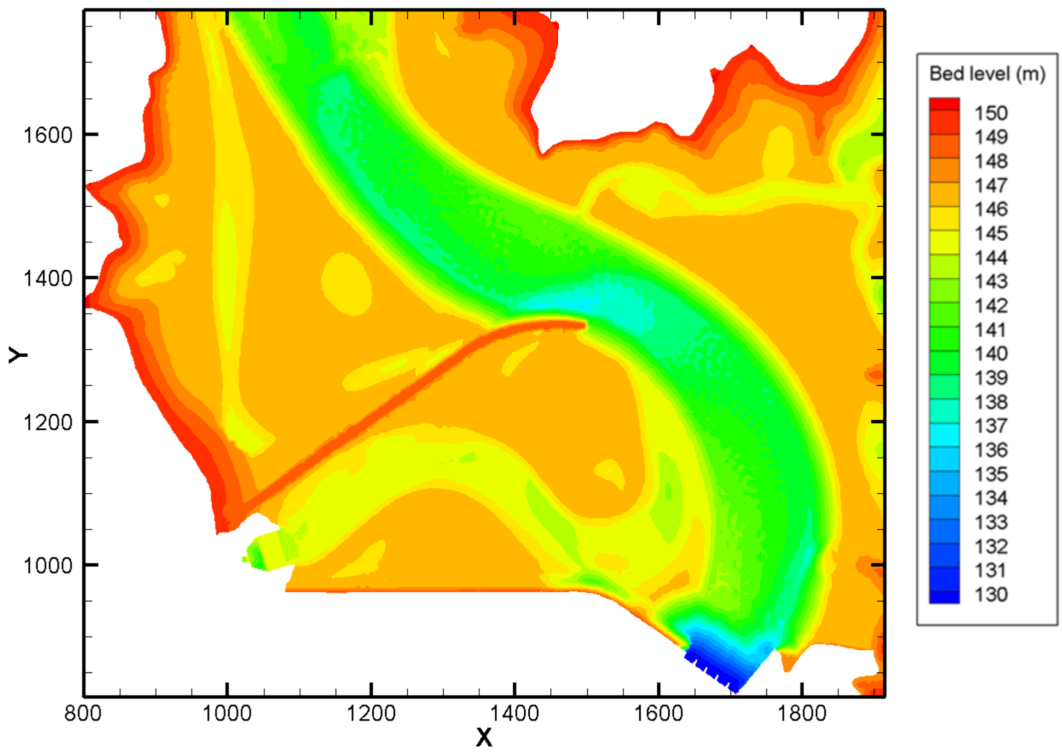

Fig. 32. Bathymetry after 10 years – Configuration D

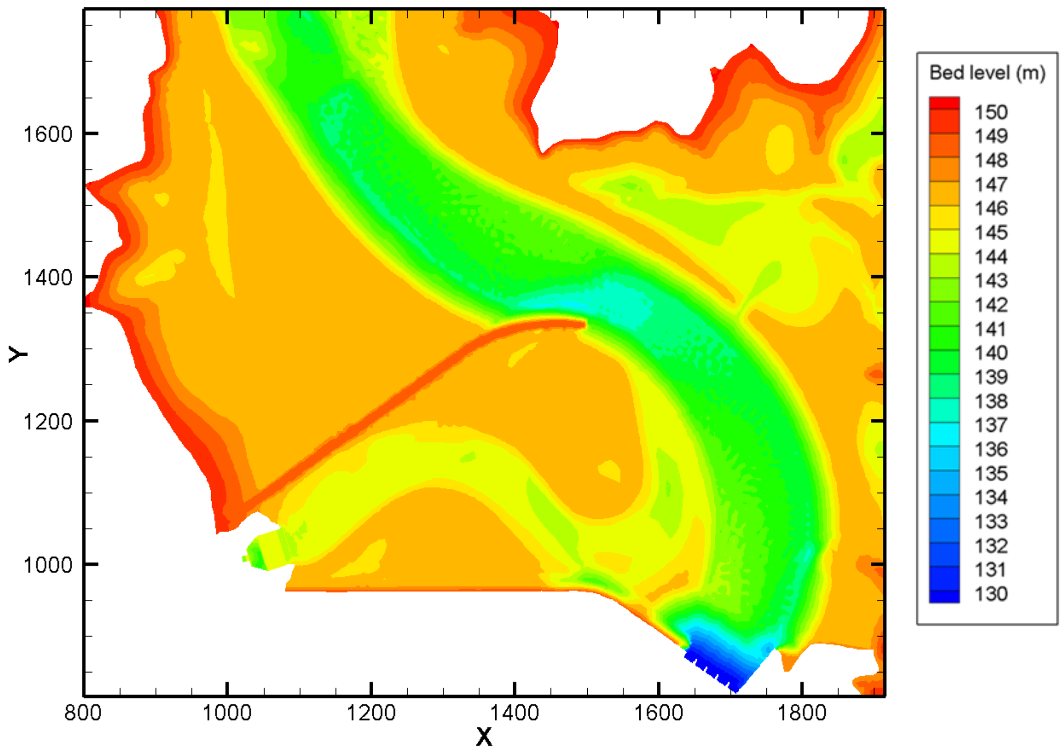

Fig. 33. Bathymetry after 10 years – Configuration E

In order to precise the time-evolution of the bathymetry during the ten-years computation period, the evolutions of the volume of sediment deposits during the runs have been computed for the three configurations. Two areas are considered: the downstream right-hand-side deposit and the whole reservoir. The results are presented in the table below:

Tabl. 6 - Evolution of the volume of sediment in the reservoir during the 10-year run

Name	RHS downstream deposit (m^3)	Rest of the reservoir (m^3)	Whole reservoir (m^3)
Ref	234 000	224 000	458 000
A	238 000	246 000	484 000
B	175 000	248 000	423 000
C	675 000	253 000	928 000
D	174 000	807 000	981 000
E	174 000	493 000	667 000

The table above presents the final result after ten years. The time evolution of the volume of sediment in the two areas considered is presented in the figures below. The first figure presents the results for the configuration A, B and C and the reference, and thus enables to see the impact of the SVIP intake, of its discharge, and of the dredging on the downstream RHS deposit.

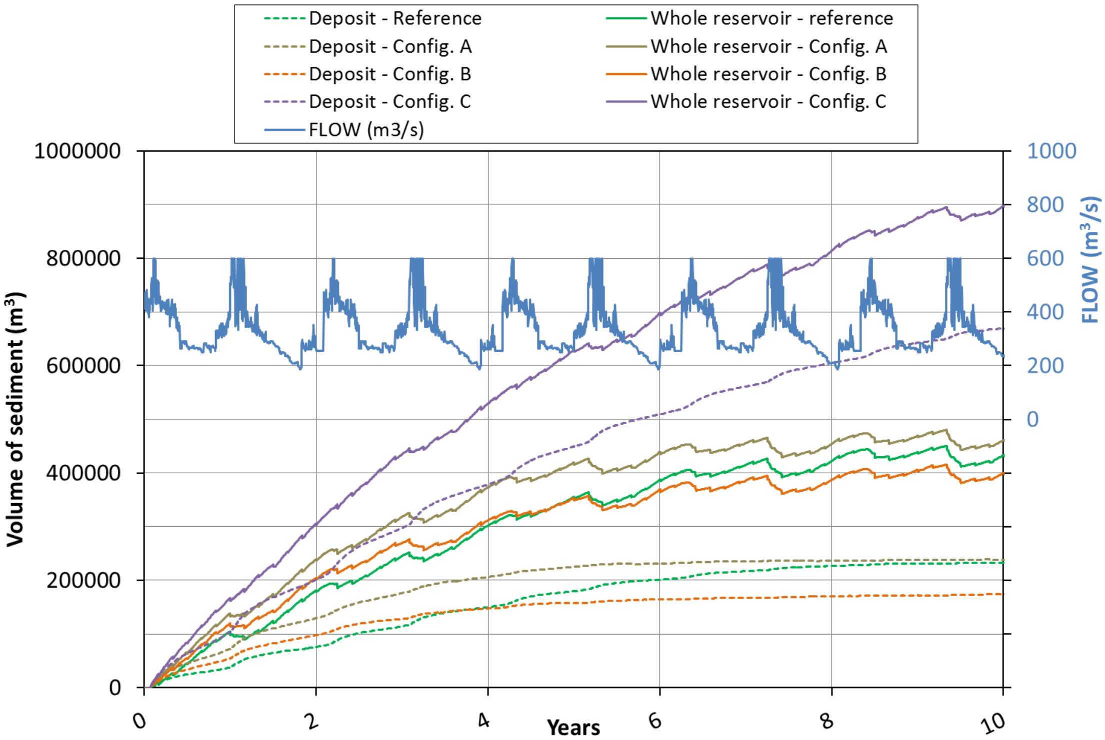

Fig. 34. Evolution of the volume of sediment on the downstream RHS deposit and for the whole reservoir

The average concentrations of sediment (clay and silt) transiting through the SVIP intake are computed, and are presented in the table below. The concentrations are averaged for the five 2-years periods that make up the 10-years hydrologic scenario.

Tabl. 7 - Sediment entering the SVIP intake

		Average concentration over 2 years periods (g/l)					Total mass entering the intake during the 10 years (t)
		0-2 years	2-4 years	4-6 years	6-8 years	8-10 years	
A	Clay (g/l)	0.096	0.105	0.108	0.109	0.108	714 000
	Silt (g/l)	*0.005*	*0.013*	*0.022*	*0.026*	*0.026*	*125 000*
B	Clay (g/l)	0.123	0.126	0.126	0.127	0.127	1 649 000
	Silt (g/l)	*0.020*	*0.025*	*0.028*	*0.030*	*0.030*	*348 000*
C	Clay (g/l)	0.116	0.119	0.124	0.128	0.131	1 622 000
	Silt (g/l)	*0.012*	*0.015*	*0.021*	*0.025*	*0.028*	*265 000*
D	Clay (g/l)	0.120	0.121	0.123	0.125	0.126	1 610 000
	Silt (g/l)	*0.018*	*0.021*	*0.026*	*0.028*	*0.029*	*322 000*
E	Clay (g/l)	0.123	0.125	0.126	0.127	0.127	1 645 000
	Silt (g/l)	*0.020*	*0.025*	*0.028*	*0.030*	*0.030*	*346 000*

The average concentrations of sediment (clay and silt) transiting through the power plant are computed, and are presented in the table below.

Tabl. 8 - Computed concentrations entering the power plant intake

		Average concentration over 2 years periods (g/l)					Mass entering the intake during the 10 years (t)
		0-2 years	2-4 years	4-6 years	6-8 years	8-10 years	
Ref	Clay (g/l)	0.149	0.148	0.148	0.148	0.148	11 864 000
	Silt (g/l)	*0.037*	*0.036*	*0.037*	*0.037*	*0.037*	*2 925 000*
	Sand (g/l)	0.0019	0.0020	0.0018	0.0021	0.0022	160 000
A	Clay (g/l)	0.153	0.152	0.153	0.152	0.152	11 710 000
	Silt (g/l)	*0.039*	*0.037*	*0.038*	*0.038*	*0.038*	*2 931 000*
	Sand (g/l)	0.0019	0.0021	0.0020	0.0024	0.0026	170 000
B	Clay (g/l)	0.155	0.154	0.154	0.153	0.153	11 369 000
	Silt (g/l)	*0.042*	*0.038*	*0.039*	*0.039*	*0.039*	*2 884 000*
	Sand (g/l)	0.0020	0.0022	0.0022	0.0028	0.0029	180 000
C	Clay (g/l)	0.155	0.153	0.154	0.153	0.153	11 305 000
	Silt (g/l)	*0.041*	*0.037*	*0.038*	*0.038*	*0.038*	*2 842 000*
	Sand (g/l)	0.0016	0.0020	0.0020	0.0022	0.0023	150 000
D	Clay (g/l)	0.152	0.151	0.153	0.152	0.153	11 222 000
	Silt (g/l)	*0.040*	*0.037*	*0.037*	*0.038*	*0.038*	*2 803 000*
	Sand (g/l)	0.0021	0.0026	0.0031	0.0029	0.0026	196 000
E	Clay (g/l)	0.153	0.151	0.153	0.153	0.153	11 263 000
	Silt (g/l)	*0.040*	*0.037*	*0.038*	*0.038*	*0.039*	*2 830 000*
	Sand (g/l)	0.0021	0.0026	0.0031	0.0029	0.0026	182 000

8.3. ANALYSIS

8.3.1. Morphodynamic impact of SVIP intake

For the reference computation, the downstream RHS deposit is almost completely filled with sediment at the end of the ten-years computation period. It means that this deposit has not reached its equilibrium state in 2016: its aggradation continues, up to a level of nearly 147 m. A channel remains on this deposit along the spur dyke. It is maintained by re-erosion of sediment during drainage of the deposit when the water level in the reservoir is lowered (for flushing operation).

When the SVIP intake is in place (all other configurations), a channel creates and maintains itself through the downstream RHS deposit. Outside this channel, the deposit tends to continue its aggradation.

At the end of the computation with the 25 m^3/s intake discharge (run with configuration A), the channel through the deposit is about 50 meters wide (30 m at the bottom and 60 meters at the top) and has a bed level of about 145 m.

At the end of the other computations (with configurations B,C, D and E), that are run with a 50 m^3/s intake discharge, the channel through the deposit is about 80 meters wide (70 m at the bottom and 100 meters at the top) and has a bed level of about 144.5 m.

The presence of the SVIP intake does not necessarily leads to lower deposition rates on the downstream RHS deposit, because more flow and thus more sediment is brought in this area when the intake is in place. With the 25 m^3/s intake discharge, the deposition rate is higher in this area than with the 50 m^3/s discharge, because the higher discharge creates stronger flow conditions than bring less deposition.

8.3.2. Impact of SVIP intake on power plant intake

When the SVIP intake is in place in the reservoir, there tends to be slightly more sediment transport at the power plant. Sediment concentrations at the power plant intake are indeed higher in the case of computations A and B than for the reference computation. This impact of the SVIP intake is stronger for coarse sediment (sand) than for fine sediment. The impact is also larger at the end of the ten-years period than at the start. This comes from the fact that the SVIP intake spreads the flow in the downstream part of the reservoir. This tends to create deposition there, and the bed level in front of the power plant is thus higher (about 1 to 1. 5 meters) when there is the SVIP intake. The higher bed level in front the power plant intake makes it slightly easier for coarse sediment to enter it. When the downstream RHS deposit is dredged (configuration C), this effect of the intake tends to disappear. In this case, sand and silt intake at the power plant are indeed not significantly higher than in the reference case. This is caused by the fact that bed sediment transiting in the channel in front of the power plant tends to be slightly finer in configuration C, because sand is more present in the whole width of the channel (while for the reference and configuration B it tends to transit on the left side of the channel).

8.3.3. Impact of dredging on SVIP and power plant intake

When a dredging is performed in the reservoir, the dredged area tends to attract sediment deposition. At the end of the ten-years computed period, the dredged area is nearly completely filled.

This deposition of sediment in the dredged areas tends to reduce the transport of sediment downstream. This reduction mostly affects silt, and not the finer sediment (clay). The most efficient location for dredging in order to reduce sediment concentration at SVIP intake is the downstream RHS deposit (computation C). Other locations of dredging have nearly no impact on sediment concentration at SVIP intake

As a consequence of deposition in the dredged areas, the effect of the dredging on the sediment transport at the SVIP intake or at the power plant intake tends to decrease with time. For example, the dredging of the downstream RHS deposit (computation C), leads to an average silt concentration at the SVIP intake of 12 mg/l for the two-years following the dredging, and of 28 mg/l after 8 to 10 years. The silt intake concentration is of 20 to 30 mg/l when there is no dredging (computation B). This indicates that if one wishes to use dredging to reduce sediment intake, the maintenance of low bed level in the dredged area will have to be performed regularly.

9. HYDRAULIC CAPACITY OF THE INTAKE

9.1. INTRODUCTION

All the analysis presented before have been performed by assuming that the target water level of the reservoir remains at 147 m. This is in accordance to the data provided by ESCOM consisting in the time-series of daily water level in the reservoir between January 2014 and January 2016. This is presented in the figure below. Other data available consists in water level from April 2015 to January 2016 at a 30 minutes time-step. This is also presented in the figure below.

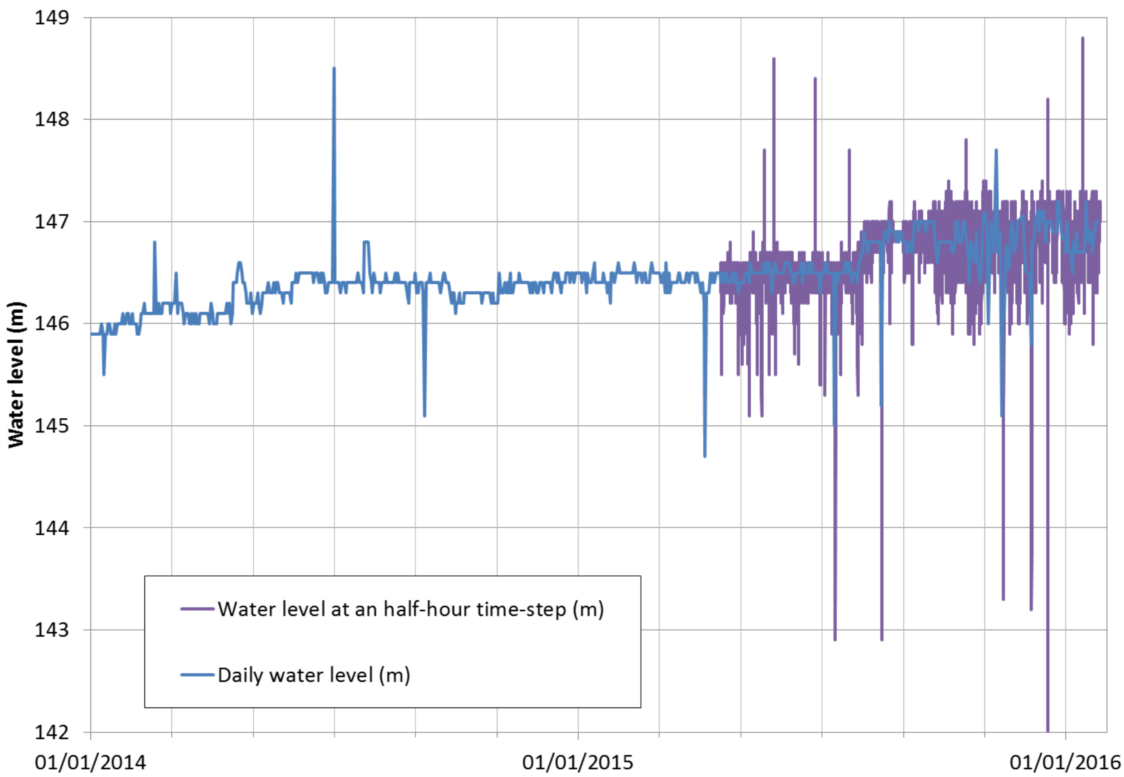

Fig. 35. Water level in Kapichira reservoir

It indicates that except during flushings, daily water level in the reservoir is kept higher than 146.2 m, and even higher than 146.5 m after August 2015. The water level at a half-hour time step indicates that there is a large variation of water level in the reservoir during the day. Even with this variation, water level in the reservoir is kept higher than 145.8 m after August 2015.

According to the Technical Feasibility Study (Draft Final Feasibility Report from October 2016), *"ESCOM operates the generators between 145.5 m and 146.5 m. This is a strict regime which should be observed in the design of the SVIP and Main 1 canal."*

Depending on the bathymetry in the reservoir, it might not be hydraulically possible for the SVIP intake to collect the whole specified discharge for low water levels in the reservoir. We precise here this particular issue.

9.2. COMPUTATION OF HYDRAULIC CAPACITY

The model is used here to determine the maximal discharge that is able to enter the water intake for a given bathymetric configuration.

For that the purpose, the model is run considering only hydraulics: sediment transport is not computed, and the bed remains fixed during the computations. A rising SVIP intake discharge (up to 50 m^3/s) is imposed. For some configurations, the hydraulic conveyance in the reservoir upstream of the intake is not high enough, which lead to supercritical flows and a limitation of the discharge that can enter the intake.

The maximum intake discharges reached for every computation are presented in the table below.

Tabl. 9 - Maximum SVIP intake discharge (m^3/s) for a given reservoir bathymetry and a given water level

	Water level			
Reservoir bathymetry	**145.5 m**	**146 m**	**146.5 m**	**147 m**
2016 measured	44.9	>50	>50	>50
2016 computed	23.1	>50	>50	>50
2018 computed with no intake	20	46.5	>50	>50
2020 computed with no intake	17.3	37	>50	>50
2022 computed with no intake	15.6	34	>50	>50
2024 computed with no intake	15	33.7	>50	>50
2026 computed with no intake	15.9	34.5	>50	>50
2018 computed with 25 m^3/s intake	6.1	27.8	>50	>50
2020 computed with 25 m^3/s intake	4.4	20	48.9	>50
2022 computed with 25 m^3/s intake	2.4	17.6	45	>50
2024 computed with 25 m^3/s intake	6	21.6	47.4	>50
2026 computed with 25 m^3/s intake	7.8	24.8	>50	>50

9.3. ANALYSIS

The results presented in the table above indicate that the maximum specified discharge of 50 m^3/s cannot reach the intake for most future bathymetric conditions when water level in the reservoir gets below 146.5 m. Even the discharge of phase 1, of 25.6 m^3/s, is not always guaranteed. These computations are considered using a fixed bed, in order to be on the safe side. In reality, the intake flow could generate erosion and deepen a channel through the bank that might enable a larger discharge to enter the intake. In the case of old deposit, the fine sediment might be partially consolidated and become very hard to erode. It is thus safer to consider fixed bed computations such as presented.

Given these results, we recommend that a regular dredging of the downstream RHS deposit is performed regularly in order to ensuring that the prescribed discharge can reach the intake.

10. CONCLUSION

10.1. GENERAL CONCLUSION

A detailed three-dimensional numerical model of Kapichira reservoir has been set up. It enables to compute flow, sediment transport (with sand, silt and clay) and bed evolution for long-term time series.

It has been calibrated on the long-term morphological evolution of the reservoir since the building of Kapichira dam.

A methodology to assess the hydro-sedimentary functioning of the reservoir taking into account the SVIP intake has then been set-up.

In a first step, different locations of the intake have been tested. The results show that the location proposed by the Technical Feasibility Consultant, i.e. downstream of the spur dyke, is preferable.

Then, longer tests (on a 10-years hydrological scenario) have been run in order to assess the morphological and sedimentary functioning of the reservoir in the future, with or without the SVIP intake. In all cases, the large sediment deposit downstream of the spur dyke continues its aggradation. When the SVIP intake is in place, a channel remains through this deposit, and enables the feeding of the intake during the 10-years period.

The impact of the SVIP intake on the intake of sediment transport at the power plant is very low. A small increase of the intake of sand is computed for some configurations. The configuration which includes dredging of the deposit downstream of the spur dyke does not show this increase.

The amount of sediment entering the SVIP intake is quite large (in average 162 000 t of clay and 26 500 t of silt per year in the best C case with dredging). The dredging of the deposit downstream of the spur dyke enables to reduce significantly the amount of the silt fraction of this sediment. But the largest part of the sediment entering the intake is composed of very fine, clayish, sediment that has a very low fall velocity. This kind of sediment can hardly settle, and its concentration at the intake is not reduced by the dredging.

Dredging of other areas in the reservoir does not reduce significantly the amount of sediment entering the SVIP intake and the power plant intake.

A worrying issue has been raised concerning hydraulic conveyance. Hydraulic computations indeed show that without maintenance of low bed levels in the reservoir upstream of the SVIP intake, the target discharge cannot be reached in the future for low water levels in the reservoir.

All results thus show that dredging of the deposit downstream of the spur dyke is very beneficial for the project. Ideally, the low bed level should be maintained permanently through continuous dredging. If not, the dredged area will progressively fill up, and the gains of the dredging will progressively disappear. Most of the effect of the dredging is lost after 4 years.

10.2. CONCLUSION ON THE DREDGING

If we compare the 10 years evolution without dredging (case B) and with dredging (case C), the impact of dredging on the quantity of silt entering the SVIP intake is a decrease of 83 000 t (24%) thanks to the dredging (SVIP discharge 50 m3/s). The dredged volume of the area reaches 560 000 m3, in average 560 000 t (with an approximate dry density of 1 t/m3 for the silt + clay mixture under water). Consequently the dredging does not appear cost-effective.

But it is important to consider:

- The safety given by the dredging to provide a permanent supply to the SVIP water intake for the range of water levels in the reservoir
- The positive impact on the sand concentration in the power intake

ESCOM told Artelia that because of the sedimentation in the reservoir they need to operate at the highest possible water level so that they still have some storage volume for peak demand and the hydraulic head is maximized. We think that their main reason for wanting to perform a large-scale dredging of the reservoir is to regain some storage and be able to run at a lower water level again if needed.

Table 9 indicates indeed that, for a 146.5 m water level and some future possible bathymetric configurations, 50 m3/s cannot be conveyed. This is more obvious when the reservoir level is 146.0 m. We can only recommend, in order staying on the safe side, to have some preventive dredging performed.

So there are two alternatives, and we recommend confirming with ESCOM their forecast for future reservoir management:

Alternative 1: if ESCOM does not perform a large scale dredging of the reservoir and continues to operate at a high water level (minimum of 146.0/146.5 m).

In this case a frequent but limited dredging of the supply channel would be the most cost-effective option. The channel widths that form in the downstream RHS deposit in the computation with configuration A, B and C (see figure below for configuration A) and its associated conveyance (see the last line of Table 9) can provide guidance for this minimal channel to be dredged

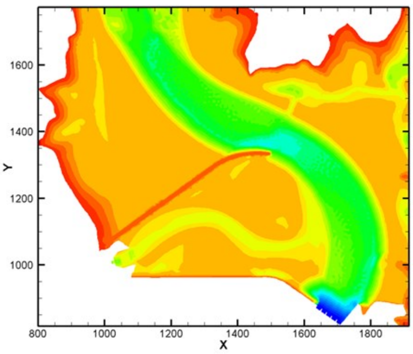

Fig. 36. Bathymetry after 10 years – Configuration A

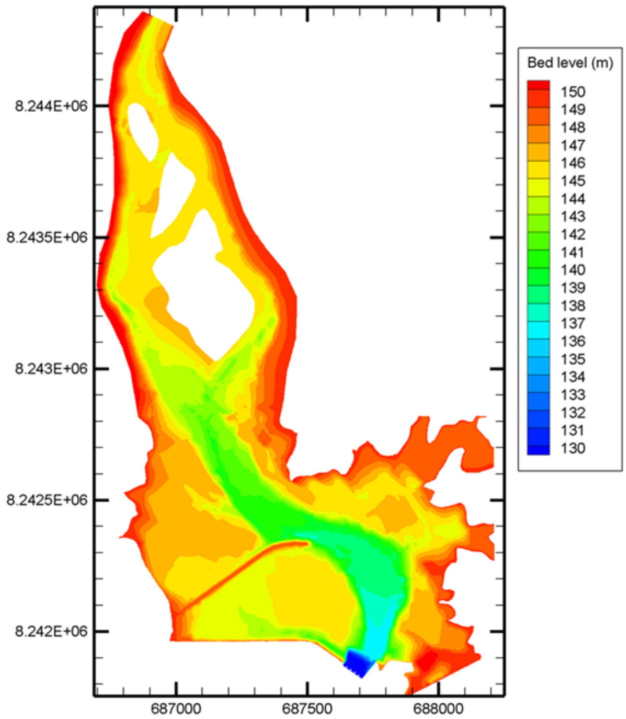

Fig. 37. Digital Terrain Model – 2016 Bathymetry

- At the end of the computation with the 25 m^3/s intake discharge (run with configuration A), the channel through the deposit is about 50 meters wide (30 m at the bottom and 60 meters at the top) and has a bed level of about 145 m.
- At the end of the other computations (with configurations B,C, D and E), that are run with a 50 m^3/s intake discharge, the channel through the deposit is about 80 meters wide (70 m at the bottom and 100 meters at the top) and has a bed level of about 144.5 m.

We thus recommend for the first step of the SVIP construction (discharge around 25 m3/s) to dredge a channel, with a width of 40 meters at the bottom and a bed level set up at 144 m to secure the intake functioning.

The location and maintenance of this approach channel is discussed below (see § 11.2.2). Artelia's recommends in conclusion:

- For the SVIP:
  - to dredge the approach channel up to the intake within the deposit at the convenient depth (see § 11.3.1) and with the appropriate width (see § 10.2 Alternative 1),
  - to set down this dredged sediment along the dam embankment
  - to monitor the channel location along the year and to dredge any deposit which could happen within this channel
- for EGENCO :
  - to keep clean the main river channel up to the spillway by periodic flushing
  - to control the water level in the reservoir during each flushing to avoid too high velocities at the spur dyke head and along the dam embankments close to the spillway abutments

Alternative 2: if ESCOM agrees that they will undertake a large scale dredging of the reservoir.

In this case, ESCOM will be able to operate at lower water levels (although the target water level will probably remain 147), which would endanger the proper feeding of the SVIP intake. In order to counteract this, the dredging (initial dredging but also maintenance dredging) should involve in priority the whole RHS downstream deposit, up to the intake. According to the Technical Feasibility Consultant, the definition of the initial dredging is the whole RHS downstream deposit at a 141 m bed level (as implemented in the model in configuration C), which is perfectly adequate. Maintenance dredging would be necessary, and its rate can be estimated from the results of the run with configuration C. The dotted violet line from figure 34 indicates that the rate of deposition in this area is of about 100 000 m3 per year. At the end of the present study, EGENCO (ex ESCOM) seems to rule out this alternative.

10.3. OTHER COMMENTS BY ARTELIA

During the flushing we consider that the reservoir level is maintained no lower than 142.9 to avoid scouring the spur dyke head.

Considering the fuse plug of the dam, an issue should be considered: did the TFS or someone else study the impact on the SVIP power intake of a functioning of the fuse plug in the right bank dam abutment? Indeed the water intake is close to this work. So what would be the bottom of the fuse plug breach in case of a one hundred or two hundred return period flood compared to the SVIP water intake setting? Is there a risk of erosion of the right bank of the reservoir around the water intake or is the presence of the rock sufficient to secure the water intake stability?

11. ANSWERS TO VARIOUS COMMENTS

11.1. ANSWERS TO EGENCO (EX ESCOM) COMMENTS

Comments on the present report were sent to the Client by EGENCO (ex ESCOM).

Answers to these comments are given below.

11.1.1. Concerning the risk to the dam

A Kapichira Reservoir Siltation Inspection Report (ESCOM, July 2003) was communicated by EGENCO (ex ESCOM) to Artelia via the Client, end of January 2017, bringing to attention some important facts concerning the hydro-sedimentary functioning of Kapichira reservoir:

- Flushing was not performed during the first 3 years of operation;
- A large sediment deposit (mostly sand) thus developed in the middle of the reservoir, blocking what should be the main channel for the flow in the reservoir;
- As a consequence, flow was diverted back to the ancient river bed (on the RHS, downstream of the spurdyke), generating erosion to the end of the spurdyke and to the main dam close to the spillway.

Artelia brings the following comments on the three different points from EGENCO:

- Model results indicate that the flow pattern is not strongly impacted by the presence of the new intake and/or by dredging,
- These results were obtained using the assumption that regular flushings will continue to be performed regularly, so that the main channel in the reservoir does not undergo major siltation again. It is this major siltation blocking the main channel that generated the diversion of the flow responsible to the damage to the structures. Regular flushings for scouring operations are included in the O&M Manual of the Kapichira sheme.

We understand that ESCOM as well as the Panel of Experts are concerned that the modification of the flow pattern in the reservoir that are induced by the SVIP intake might lead to higher risk of a repeat of the river diversion that occurred in 2003. We think, based on results of the numerical model, that the SVIP intake does not increase this risk. The figure below shows flow patterns (for a river inflow of 600 m3/s) computed for different bathymetric configurations, and with or without the SVIP intake. When the SVIP intake is in place, there is indeed a flow along the dam, but the current is not strong (below 0.5 m/s) and is very different (opposite direction) than the one that caused erosion in this area in 2003.

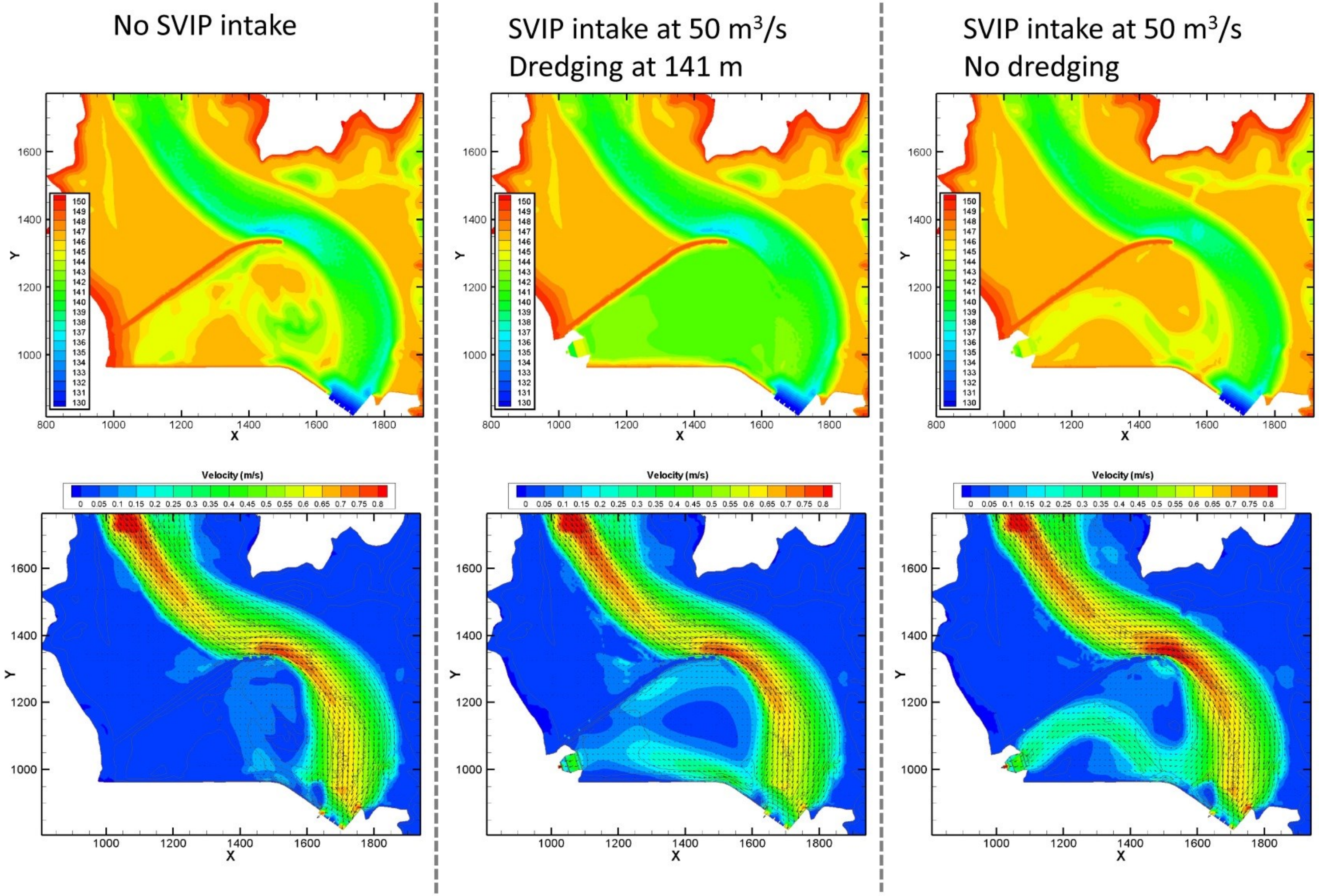

Fig. 38. ***Bathymetry and velocities with or without SVIP intake***

11.1.2. Concerning the dredging

EGENCO says: “Note also that in terms of dredging of the Reservoir at Kapichira, ESCOM (EGENCO now) is already procuring a dredger for the same. However, ESCOM (EGENCO now) does not intend to dredge in the “dead” area of the Reservoir (for obvious safety and operational reasons), but only in areas in the main channel”.

Artelia notes that the dredging that will be effectively performed, according to EGENCO, seems to be very different from the dredging planned in the Technical Feasibility Study (see Final Feasibility Report of December 2016, page 13-3). In the TFS report it is written indeed: “ESCOM has a plan to dredge this area to 143 m a.m.s.l., which is low enough for the normal operation of the intake structure. TFS team has discussed with ESCOM engineer and confirmed that the dredging project is well underway and is expected to be implemented in the second half of next year”.

With the new position of ESCOM on the dredging operations, the only option left to ensure a smooth conveyance of water to the SVIP intake, is a channel as proposed with Alternative 1 in the conclusion of our final report (see above §10.2 Alternative 1). Alternative 1 is appropriate considering EGENCO wishes for the Kapichira reservoir operation. The use of such a channel (against a full dredging of the area) would limit the risk that a catastrophic diversion of the main channel in the reservoir (like in 2003) does happen again. Again, we reassess that the use of regular flushing is the best and necessary way to ensure that the main flow stays in the main channel, which is the normal functioning of the Kapichira Reservoir according to its initial design and O&M Manual.

11.1.3. Construction of a new dyke along the access canal to the SVIP intake

According to the Visit Report by Panel of Experts (§ 6.2 EGENCO’s Comments on SVIP Intake location), “EGENCO expressed concern on the proposed location of SVIP intake on the downstream side of the dyke” (the existing spur dyke). The Visit Report says that this concern “can probably alleviated by construction of a dyke on the left side of the approach channel of the intake of SVIP more or less parallel to the existing dyke”.

Considering the flow pattern and velocities, as shown in figure 38 above, it appears that there is no need for such work. Artelia does not recommend building another dyke on the left side of the SVIP intake access channel. There is no need for this work but it is supposed that no large sediment deposit blocks the main channel (like in 2003), thanks to regular flushing in the reservoir, and/or dredging operations.

11.2. ANSWERS TO THE PANEL OF EXPERTS COMMENTS

11.2.1. Reservoir Level Operation and approach channel setting

According to the Panel of Experts, observations (Visit Report by Panel of Expert, page 13, Target Reservoir Operating Level) “the present target level at 147 m is 2.5 m higher than the recommended design water level, and presumably is an encroachment on the free board” and “ Encroachment on freeboard is a safety concern specially for a dam which is designed for Q100 and that too with fuse plug being activated at Q100”. With this 147 m water level, the fuse plug freeboard is only 1 m. If, as a consequence of the Panel of Expert fuse plug concern, the water level in the reservoir is lowered in the future, it will be however necessary to maintain the same velocities for the SVIP intake access channel of Alternative 1.

According to the model computations, the convenient velocity in the SVIP intake access channel for a level variation between 147 and 146.5 m is:

- between 0.25 and 0.35 m/s for the 25 m3/s discharge,

- between 0.25 m/s to 0.4 m/s for the 50 m3/s discharge.

To maintain these values at a possible lower reservoir level, it will be necessary to lower the same way the access channel bottom to maintain the same characteristics of velocity. More information is given below concerning the channel approach dredging (answers to the comments from the Bank).

11.2.2. Guide banks on either side of the intake or free approach channel

The POE considers that it is not advisable to let a current (even with low velocity) along the dam embankment. Artelia agrees that it is more convenient to have an approach channel fixed not along the dam embankment but in the middle of the deposit area. We have found that, when there is no dredging of the whole area between the spur dyke and the dam, the approach channel tends to be maintained within the deposit. So Artelia thinks that if the approach channel is started by a dredging in the middle of the current deposit, it will be maintained by the flow towards the intake. The dredged material of the approach channel could be dumped along the dam embankment to limit the possibility of the creation of a return current along this embankment. Besides, if at a moment a deposit is observed in this approach channel, due to some reason like a temporary stopping of the SVIP discharge, and if, as a consequence, a return current is observed along the dam embankment, the velocities of this current are so low that there is absolutely no risk for the embankment stability. If after the starting up again of the SVIP intake, the current is maintained along the dam embankment, instead of along the previous alignment, it will be easy to dredge the deposit which prevents the flow to be redirected towards the intake and to restore this channel.

The creation of the guide banks is possible but will be very costly. The difficulties are as follows:

- We do not know how deep this work will have to be founded to avoid its lowering into the silt material. Some geotechnical survey would be necessary to know this depth according to the measured bearing capacity to various depths
- Anyway the volume of material will be important as the length of the guide banks will be also important to reach a location of the nose of the each guide bank near the deep channel of the river (but not too close to avoid a risk of erosion of theses noses by the flow during the flushing)
- The works will have to be done under water with a lot of dredging (not only the approach channel but also the excavation for the guide banks construction)

In conclusion we recommend:

- For the SVIP:
  - to dredge the approach channel up to the intake within the deposit at the convenient depth (see § 11.3.1) and with the appropriate width (see § 10.2 Alternative 1),
  - to set down this dredged sediment along the dam embankment
  - to monitor the channel location along the year and to dredge any deposit which could happen within this channel
- for EGENCO :
  - to keep clean the main river channel up to the spillway by periodic flushing
  - to control the water level in the reservoir during each flushing to avoid too high velocities at the spur dyke head and along the dam embankments close to the spillway abutments

11.3. ANSWERS TO THE WORLD BANK EXPERTS COMMENTS

11.3.1. Volume of sediment to be dredged

Question: What is the volume of sediment to be dredged initially to secure the initial dredging? According to table 9, and considering a commissioning of the scheme by 2022, the intake discharge would only be 15.6m3/s, which seems to show an initial dredging would be required to secure 25m3/s.

Answer from Artelia: Considering a present bed level of 147 m, an intake channel 40 m wide at the bottom and 3 meters deep, bank slope for the channel of 1 per 5, the volume to be dredged is presented in the table below, depending on the minimum operating water level target in the reservoir.

Tabl. 10 - Volume of sediment to be dredged

Water level (m)	Volume to be dredged (m3)	Channel bottom to be maintained (m)
147	99 000	144
146.5	121 000	143.5
146	144 000	143
145.5	169 000	142.5
145	195 000	142
144.5	223 000	141.5

11.3.2. Maintenance dredging

Question: With which frequency and volume a maintenance dredging should be performed to secure a 25m3/s intake discharge? And a 50m3/s intake discharge?

Answer from Artelia: The channel should maintain itself without need for significant dredging according to the model results. This sustainability will have to be checked carefully and regularly, in case an unforeseen event (for instance, massive sediment influx in the reservoir, lack of regular flushing, closure of the SVIP intake for a long period …) generates some deposit in the channel. The fact that the channel is stable indeed supposes that the discharge flows continuously through SVIP intake. If this is not the case (for instance if the SVIP intake is closed at night), sediment deposition will occur at the entrance of the channel dredged in the deposit. It might be necessary to perform complementary computations once the precise schedule and functioning of the irrigation scheme is known.

11.3.3. Uncertainties and assumptions

Question: We acknowledge that there are a number of uncertainties and assumptions, including the use of a fixed bed and not take into account channel erosion by the intake flow. Is there anything that can be said on the quantitative impact of this assumption?

Answer from Artelia: the model is mainly run with a movable bed all the time, so the access channel to the SVIP intake is dug and shaped by the flow along the computations (along the years). We just made some additional hydraulic studies to check the access channel capacity in various conditions of water level and various conditions of bathymetry (previously computed with the movable bed model) and SVIP discharge. The use of a fixed bed in this case enable to stay on the safe side (as explained in § 9.3 above).uy