Case Study

Modeling of Sediment Management for the Lavey Run-of-River HPP in Switzerland

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Abstract: Reservoir sedimentation hinders the operation of the Lavey run-of-river hydropower plant (HPP) on the Rhone River in Switzerland. Deposits upstream of the gated weir and the lateral water intake reduce the flood release capacity and entrain sediments into the power tunnel. Past flushing operations of the relatively wide and curved reservoir have been inefficient. To improve sediment management, the enhanced scheme Lavey+ with an additional water intake and a training wall for improving flushing was set up. The performance of the enhancement project was tested on a physical model. For its calibration, sediment transport, deposition, and flushing of the present scheme were investigated and compared with prototype measurements. The enhanced scheme was then analyzed in detail to define the flushing discharge and duration, and define the gate operation to ensure maximal erosion of deposits with minimal water loss.
DOI: 10.1061/(ASCE)HY.1943-7900.0000505.

CE Database subject headings: River systems; Reservoirs; Sediment; Switzerland; Hydro power; Power plants.

Author keywords: Run-of-river HPP; Physical modeling; Reservoir sedimentation; Flushing.

Introduction

Present Scheme

Construction of the Lavey run-of-river hydropower plant (HPP) on the Rhone River in Switzerland [Figs. 1 and 2(a)] was completed in 1949. The current existing water intake (subscript TE) on the right river bank supplies the underground power house with a maximum discharge (Q_{TE}) of 220 m^3/s. The head varies between 34 and 42 m. Three 31 MW turbine units produce approximately 400 $GWh/m^3/year$. At El. 435, the weir has three 13-m wide openings equipped with drum gates. Between gates 2 and 3, a submerged training wall with its crest at El. 444 allows equilibrated flow patterns for operation mode and for flood events.

The Rhone River at Lavey has a mean annual discharge (Q_a) of 180 m^3/s. Approximately 50% of the water volume is associated with discharges (Q_R) between 100 and 200 m^3/s. Flood events resulting from heavy rainfall, snow, and glacier melt generally occur in August. The annual flood (HQ_1) is approximately 500 m^3/s. The design flood corresponds to the probable maximum flood (PMF), which authorities in 2008 increased from 1,750 m^3/s to 1,915 m^3/s. For technical reasons relating to trash rack blockage, turbining is stopped at discharges (Q_R) greater than 800 m^3/s.

As is often the case for run-of-river HPPs, the Lavey scheme is affected by continuous reservoir sedimentation. According to prototype observations, bed load transport starts at flows greater than 300 m^3/s, producing 80% of total sediment transport to the reservoir of approximately 25,000 $m^3/year$. During the flood event in October 2000, inundations at the dam site at Lavey were narrowly avoided. Measurements after the event revealed significant deposits along the entire 3-km length of backwater, which primarily originated from the bed load transport. Significant sediment deposits up to 8 m thick were detected, especially at the inner river bend of the curve immediately upstream of the weir. In the past, regular flushing was necessary to maintain reservoir volume and to reduce the inundation risk during floods. Every 2 to 13 years, flushing was conducted when the control transversal profile upstream of the weir showed excessive deposition as in 1968, 1969, 1982, 1985, 1990, 1997, and 2005. The efficiency of these flushing operations was always quite low and even decreased over time because a concentrated stream developed along the outer bank. Sediments were only eroded in front of the weir and near the intake, whereas substantial deposits at the inner side of the curve were not removed.

Enhanced Scheme Lavey+

In 2007, the owner of the HPP investigated the enhancement project, Lavey+ [Fig. 2(b)]. In addition to improving sediment management, a parallel supply tunnel will reduce head losses in the existing power tunnel. An additional turbine increases the flexibility of power production. The additional water intake (subscript TN) with a design discharge (Q_{TN}) of 140 m^3/s is located 37 m upstream of the existing intake on the right river bank. The lengthened training wall [D in Fig. 2(b)] and the rounded inner bank create a continuously constricting flushing channel toward gates 1 and 2. These constructive measures should avoid sediment entrainment toward the intakes and promote efficient flushing. To adequately supply the new water intake, the crest of the training wall at El. 444 is lowered to El. 443 for the 44-m long notch [E in Fig. 2(b)]. Behind the training wall, a 13-m wide secondary

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Note. This manuscript was submitted on April 29, 2011; approved on September 14, 2011; published online on September 19, 2011. Discussion period open until September 1, 2012; separate discussions must be submitted for individual papers. This paper is part of the ***Journal of Hydraulic Engineering***, Vol. 138, No. 4, April 1, 2012. ©ASCE, ISSN 0733-9429/2012/4-340–347/$25.00.

Fig. 1. Reservoir of the Lavey HPP with water intake (A) and weir (B) on the Rhone River [photo courtesy of the Services Industriels of the City of Lausanne (SIL)]

flushing channel [J in Fig. 2(b)] passes in front of the two water intakes to evacuate the settled fine sediment on the outer bend.

As Knoblauch (2006) discusses, flushing removes sediment deposits from the reservoir by using the shear stress of flow. Other measures such as dredging or constructive measures at the intakes have also been evaluated for removing sediment. However, periodical flushing by natural river flow has many advantages from a river-morphological and economic point of view. It is efficient when the water surface is drawn down to a minimum level and sufficient discharge is available, thereby generating high flow velocities. Knowing the grain fraction, contained substances, compactness, and river bed and bank structures in the reservoir helps in selecting the suitable threshold values for flushing operations. These characteristics and prototype monitoring data showing sedimentation and flushing processes were available for the current study. Flushing at low water level and sluicing through the gates at normal reservoir operation level were optimized for the enhanced Lavey+ HPP.

Hydraulic Model

Experimental Set-Up

To optimize the enhancement project, especially the flushing procedure, a hydraulic model [Figs. 3(a) and 3(b)] was built at the Laboratory of Hydraulic Constructions (LCH) of the Ecole Polytechnique Fédérale de Lausanne (EFPL). The approach flow conditions in the river during power production and flood periods, sediment transport, sediment deposition, and flushing were analyzed. The model was operated in accordance with the Froude similarity with a length scale ratio of 1:40, reproducing some 500 m of the Rhone River (that is, 350 m upstream and 150 m downstream of the Lavey weir). Hydraulic elements such as the dam and the water intakes were reproduced with PVC and integrated in the mortar topography and river morphology.

The Rhone River discharge and the flow extractions at both water intakes were individually measured with electromagnetic flowmeters to $\pm 0.5\%$ accuracy. A mini-echo sounder (Ultralab UWS, General Acoustics, Kiel, Germany) measured the evolution of the river bed during the experiments at an ultrasound frequency of 1 MHz, allowing ± 1 mm accuracy (Kantoush et al. 2008). The UWS was installed on a linear single beam support and recorded eight upstream and two downstream cross sections of the present scheme, and 10 upstream and two downstream cross sections for Lavey+ scheme, which covers 250 m of the reservoir upstream from the weir [Figs. 3(c) and 3(d)]. These measurements were transversally performed for every 2 m of the cross section.

Müller et al. (2010a, b) provides a detailed analysis of approach flow and flood behavior of the present scheme. Several numerical simulations have been conducted to define the border conditions and to optimize the layout of the HPP (Bieri et al. 2010).

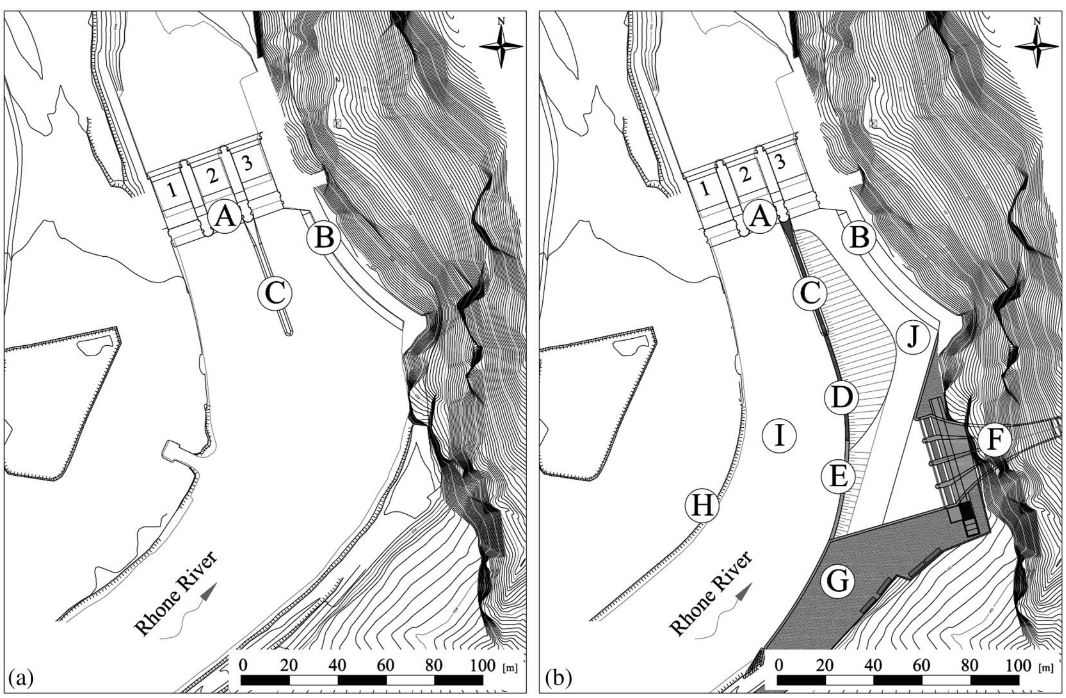

Fig. 2. (a) Present scheme; (b) enhanced scheme Lavey+; A is the weir with gates 1, 2, and 3; B, HPP water intake; C, the existing training wall; D, the lengthened training wall; E, the wall segment with notch; F, the new water intake; G, the maintenance platform; H, the rounded inner bank; I, the main flushing channel; and J, secondary flushing channel

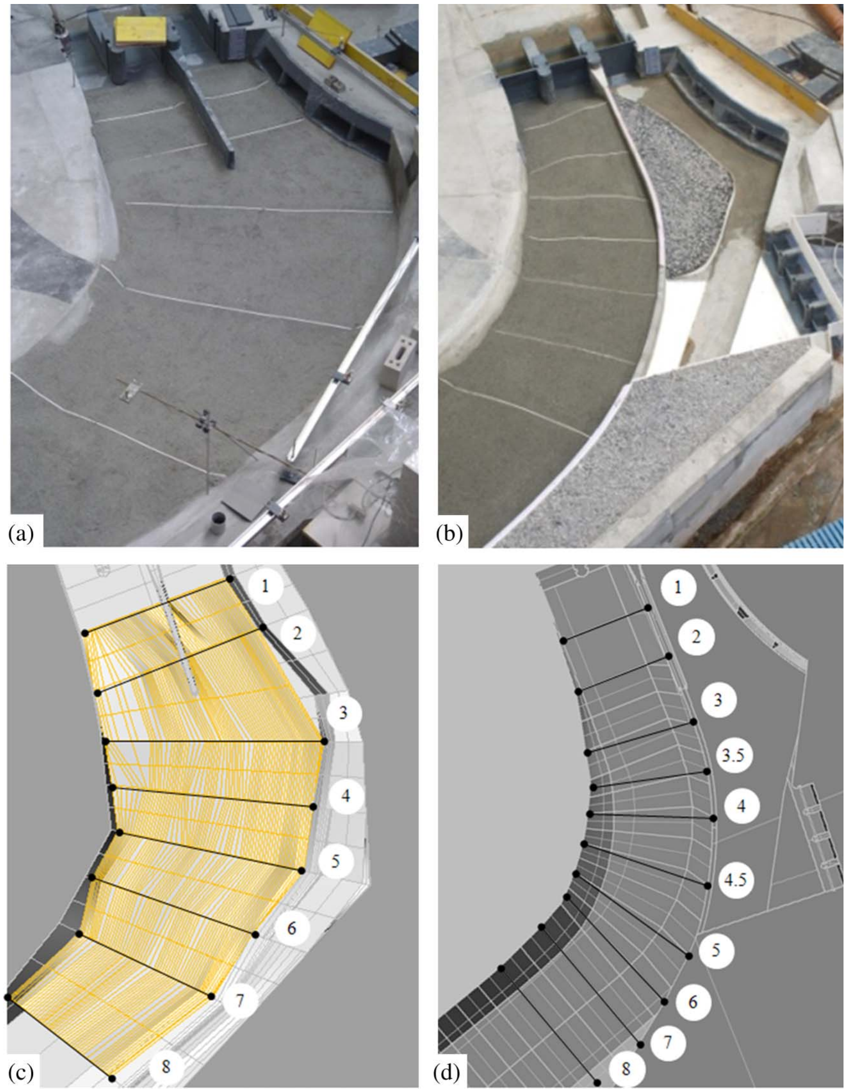

Fig. 3. Hydraulic model of the reservoir and considered cross sections upstream of the existing weir: (a, c) the present scheme; (b, d) the enhancement project Lavey+

Sediment Characteristics and Supply

For experiments with sediments, Froude and sediment transport similitudes were considered (Julien 2002). The prototype bed load had a mean diameter (d_m) of 3.0 cm and 30% fraction diameter (d_{30}) and 90% fraction diameter (d_{90}) of 0.9 cm and 8.0 cm, respectively. The Shields diagram in Fig. 4 reveals sediment behavior at an upstream cross section 250 m from the weir, as a function of boundary Reynolds number (R^*) and dimensionless shear stress (τ_*). Sand and smaller fractions $(d \leq d_{30})$ are in motion for the entire operation flow range $(Q_R \leq 800\ \mathrm{m}^3/\mathrm{s})$. Discharges up to $300\ \mathrm{m}^3/\mathrm{s}$ mobilize gravel and sand and contribute to most of the bed load transport. During floods, the Rhone River even transports coarse material. During the 2005 flushing in which Q_R was $90\ \mathrm{m}^3/\mathrm{s}$, the critical shear stress in the investigated river reach was exceeded for the entire grain size distribution. The bed load was simulated in the hydraulic model with noncohesive sand $(d_{30} = 0.33\ \mathrm{mm}, d_m = 0.8\ \mathrm{mm}, d_{90} = 2.0\ \mathrm{mm})$, in accordance with the Shields criterion for operational and flushing flow rates, as Fig. 4 shows.

During test runs, sediment was manually supplied at the upstream end of the model. For the initial filling process, to achieve the reference state of deposits before flushing, the input of excessive material accelerated the formation of the river bed. In the second step, the input of sediment was adjusted to the transport capacity of the upstream reach of the Rhone River, which was obtained by two-dimensional (2D) numerical modeling by using the Smart and Jaeggi bed load relationship (Jaeggi 1984). For systematic flushing tests, the reference bathymetry was reestablished in the model without resimulating the entire filling and stabilization process.

Model Validation

Sediment transport, deposition, and flushing behavior in the prototype had to be reproduced to validate sediment-related processes in the model. The well-documented 2005 flushing was chosen as the reference case. The bathymetry of the reservoir before the operation had been measured by echo sounding [Fig. 5(a)]. Compared with the initial reservoir geometry before impounding in 1949, the deposits before flushing (V_0) reached $72{,}000\ \mathrm{m}^3$. To achieve a similar bed morphology in the hydraulic model, the sedimentation process had to be accelerated and simplified. In a continuous test lasting eight days (corresponding to approximately two months in reality), a series of low flow discharges and flood events was simulated, as Fig. 6 shows. During this time, a constant turbine operation was maintained with Q_{TE} at $220\ \mathrm{m}^3/\mathrm{s}$. During the initial

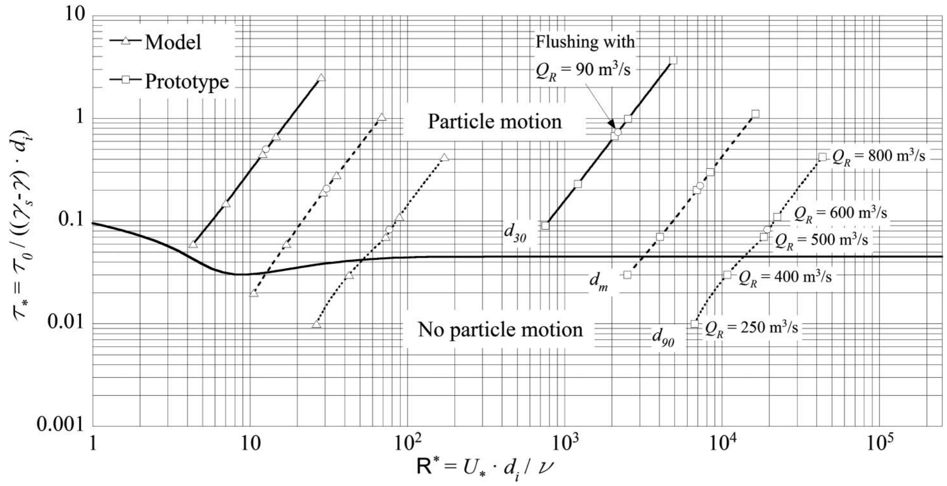

Fig. 4. Shields diagram for operation and flushing discharges; comparison of sediment behavior in the prototype and the hydraulic model at the cross section located 250 m upstream of the weir [τ_* = dimensionless shear stress; τ_0 = shear stress; γ_s = specific weight of sediment (2,650 $\mathrm{N/m^3}$); γ = specific weight of water (1,000 $\mathrm{N/m^3}$); d_i = particle diameter of fraction i; R^* = boundary Reynolds number; U_* = shear velocity; and ν = cinematic viscosity of water (10^{-6} $\mathrm{m^2/s}$)]

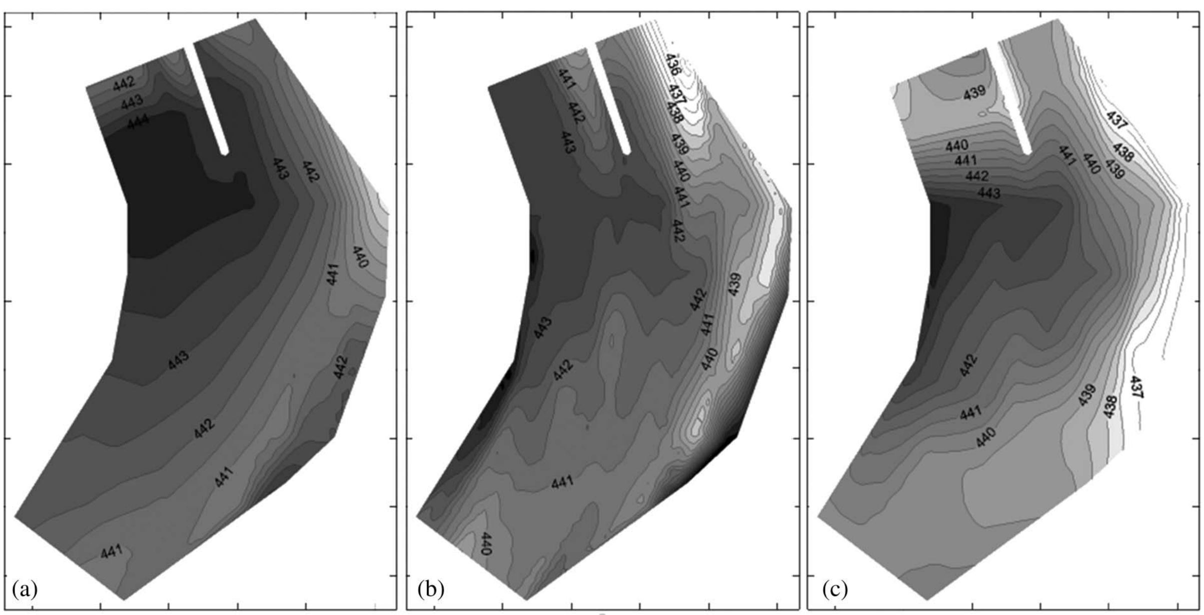

Fig. 5. Bed elevation for the present configuration before flushing: (a) in the prototype (2005); (b) in the model; (c) after flushing in the model

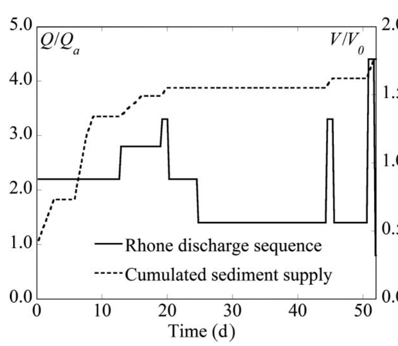

Fig. 6. Sequence of relative discharge (Q/Q_a) and relative sediment supply (V/V_0) applied in hydraulic modeling to attain the 2005 reference bathymetry in the reservoir before flushing; Q_a = mean annual discharge of the Rhone River; and V_0 = volume deposited before flushing

filling process, for discharges (Q_R) that are two to three times greater than the annual mean discharge (Q_a) of 180 $\mathrm{m^3/s}$, sediment supply was maintained much greater than that obtained by the 2D numerical modeling of the upstream Rhone River reach. Then, several flood events with corresponding sediment supply were subsequently conducted to achieve the equilibrium bed morphology (i.e., sediment input is equal to output). Considering that between 1997 and 2005 several flood events occurred with peak flows in which the Q_R ranged between 400 and 1,250 $\mathrm{m^3/s}$, two flood events in which the Q_R was 600 $\mathrm{m^3/s}$ and one flood event in which the Q_R was 800 $\mathrm{m^3/s}$ were simulated in the model. The simulations had satisfying morphological results. After every event, eight cross sections upstream of the weir were measured and compared with the prototype measurements until achieving the 2005 bathymetry.

The characteristic bed morphology with significant sediment deposits higher than El. 444 at the inner side of the bend and a scoured channel at the outer bank were achieved by simulating

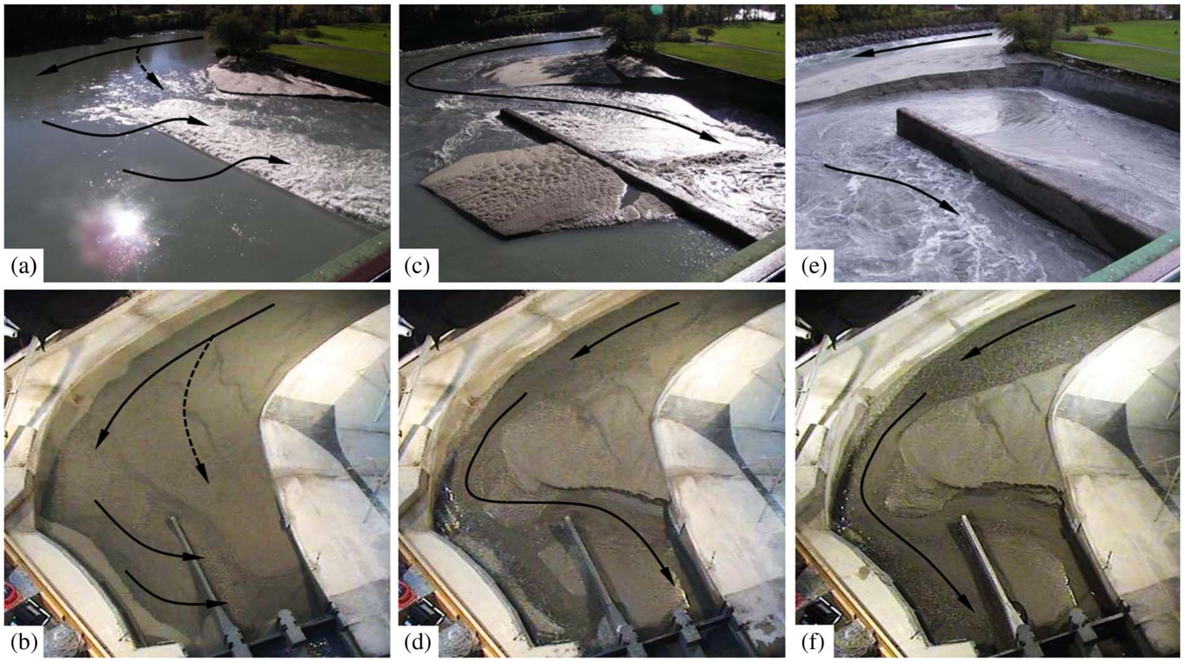

Fig. 7. Chronology of the 2005 flushing ($Q_R = 90\ \mathrm{m^3/s}$) in the prototype (a, c, e) and the hydraulic model (b, d, f): (a, b) after 10 min with gate 2 open; (c, d) after 110 min with gates 1 and 2 open; (e, f) after 210 min with all gates open

three flood events. However, the final bed geometry had a deposit volume (V_0) of 58,000 $\mathrm{m^3}$ [Fig. 5(b)], which is less than the prototype measurement of 72,000 $\mathrm{m^3}$.

The 2005 prototype flushing (Fig. 7) was performed with Q_R at 90 $\mathrm{m^3/s}$ without turbine operation. Center gate 2 was slowly opened over a period of 30 min. As the water level decreased, the flow headed along the outer river bank toward the intake; it then evacuated through the open gate by overflowing the training wall. After gate 1 had also been opened, the water level decreased and the water supply from the inner side of the bend was therefore completely blocked. Flow from upstream bypassed the sediment deposits located at the inner bend, thereby eroding it only laterally. By opening gate 3 on the right bank, sediments between the water intake and the training wall were finally eroded. The same discharge and operation mode were adopted in the laboratory test, as Fig. 7 shows. Visual comparison of the flushing indicates good agreement between the prototype and the model results. The erosion processes correspond phenomenologically and temporally. After 3 h of flushing in the model (approximately 18 h in reality), only 10,000 $\mathrm{m^3}$ of sediment were removed [Fig. 5(c)]. This low flushing performance was also observed visually during the 2005 flushing in the prototype, but it was not quantified. Only a few sediments settled downstream of the weir and some were even remobilized later.

However, a similar bed morphology could be reproduced, even if simulated volumes before flushing were less than the prototype measurements. The fact that the main flushing phenomenon was correctly simulated confirmed the chosen simplified experimental approach with regard to boundary conditions and sediment supply for the simulation of future scenarios.

Flushing Concept for Enhanced HPP

Overview

The key element of the Lavey+ sediment management strategy is the lengthened training wall connecting the pillar between gates 2 and 3 to the right river bank 100 m upstream of the new water intake [D in Fig. 2(b)]. The geometry of this wall was optimized to limit sediment entrainment into the zone in front of the two water intakes and to guarantee optimal approach flow to the intakes. However, the main objective regarding sediment management was to create a channel at the inner bend that is able to increase bed load transport toward the weir gates while allowing efficient flushing operation. Thus, the cross section of the channel narrows progressively from 60 m upstream of the training wall to 30.5 m at the weir. The training wall allows concentration of the flow at the inner bend during flushing.

Three specific parameters affecting flushing performance were investigated in the hydraulic model: the initial bed geometry (i.e., reference bathymetry); the gate opening sequence; and the flushing discharge. The owner of the HPP defined the main operation rules regarding gate opening procedures and the overall flushing duration (t_F) of 24.0 h.

The mobile bed evolution during flushing was measured by the mini-echo sounder during experiments at all up- and downstream cross sections. Sediment balance and output could be defined at different stages of the flushing operation ($t = 1.5$ h, 3.0 h, 6.0 h, 12.0 h, and 24.0 h).

Sediment Deposition and Monitoring

To establish a reference bathymetry before flushing for the configuration of Lavey+, the experimental sequence applied during model calibration was adopted (Fig. 6). In the first step, a filling and equilibration period with constant discharge was performed. In the second step, three flood events with continuous turbining ($Q_{TE} = 160\ \mathrm{m^3/s}$ and $Q_{TN} = 75\ \mathrm{m^3/s}$) were performed. As for the present scheme, the corresponding equilibrium state in the retention zone generated more deposits on the inner bank and concentrated the flow at the outer bank because of the bend effect. The sequence with flood events applied to the present state led to strong erosion along the training wall, which attained the original bed geometry. The inner bank was not influenced by the flushing because the flow was concentrated at the outer side of the bend in the

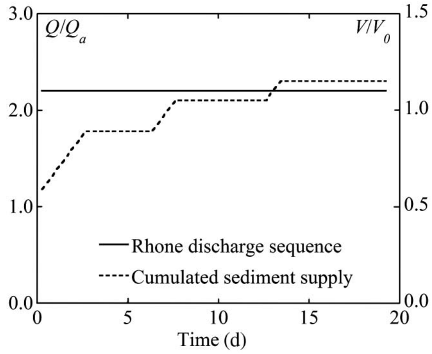

Fig. 8. Sequence of relative discharge (Q/Q_a) and relative sediment supply (V/V_0) applied in hydraulic modeling to attain Lavey+ reference bathymetry in the reservoir before flushing; Q_a = mean annual discharge of the Rhone River; and V_0 = volume deposited before flushing

channel that is created by floods. Therefore, sediments could not be evacuated during flushing.

Because the previous passage of floods affects the flushing performance, a second scenario of bed evolution was tested in the hydraulic model. The Rhone River was exposed to a sedimentation process without periods of major floods but with a constant discharge $(Q/Q_a = 2.2)$, as Fig. 8 shows. The resulting equilibrium bathymetry showed transversally uniform bed elevation in the Rhone River [Figs. 9(a) and 9(b)]. The relevant flow section for flushing is consequently larger and erodes the deposits of the inner bend laterally and frontally. Therefore, it is recommended that flushing operations be initiated after periods without major flood events.

Flushing and Sluicing

Two indicators were used to evaluate the performance of various flushing scenarios performed at different discharges. The V_F/V_0 ratio, which is the cumulated flushed sediment volume (V_F) divided by the initial deposited volume (V_0) before operation, indicates the flushing performance at any time in the experiment. The V_F/V_{WL} ratio, which is the cumulated flushed sediment volume (V_F) divided by the cumulated volume of water losses $\left(V_{WL} = \int Q\,dt\right)$, is the operation efficiency. Compared with the present state, initial deposits in the channel before flushing were lower $(V_0 = 57{,}700\ \mathrm{m}^3)$ because of reservoir splitting by the training wall.

The first tests revealed greater flushing performance with an initial opening of center gate 2, giving a total flushing of V_F/V_0 ratio of 44%, compared with a V_F/V_0 of 29%, for when left gate 1 is opened first. Flushing sequences in which t_F is 24.0 h were consequently divided into five cycles, with center gate 2 and left gate 1 opened alternately for 1.5, 1.5, 3.0, 6.0, and 12.0 h. Such alternating gate operation is recommended because of the dynamic flow effects during repeated raising and lowering of the water level in the upstream river reach. To determine

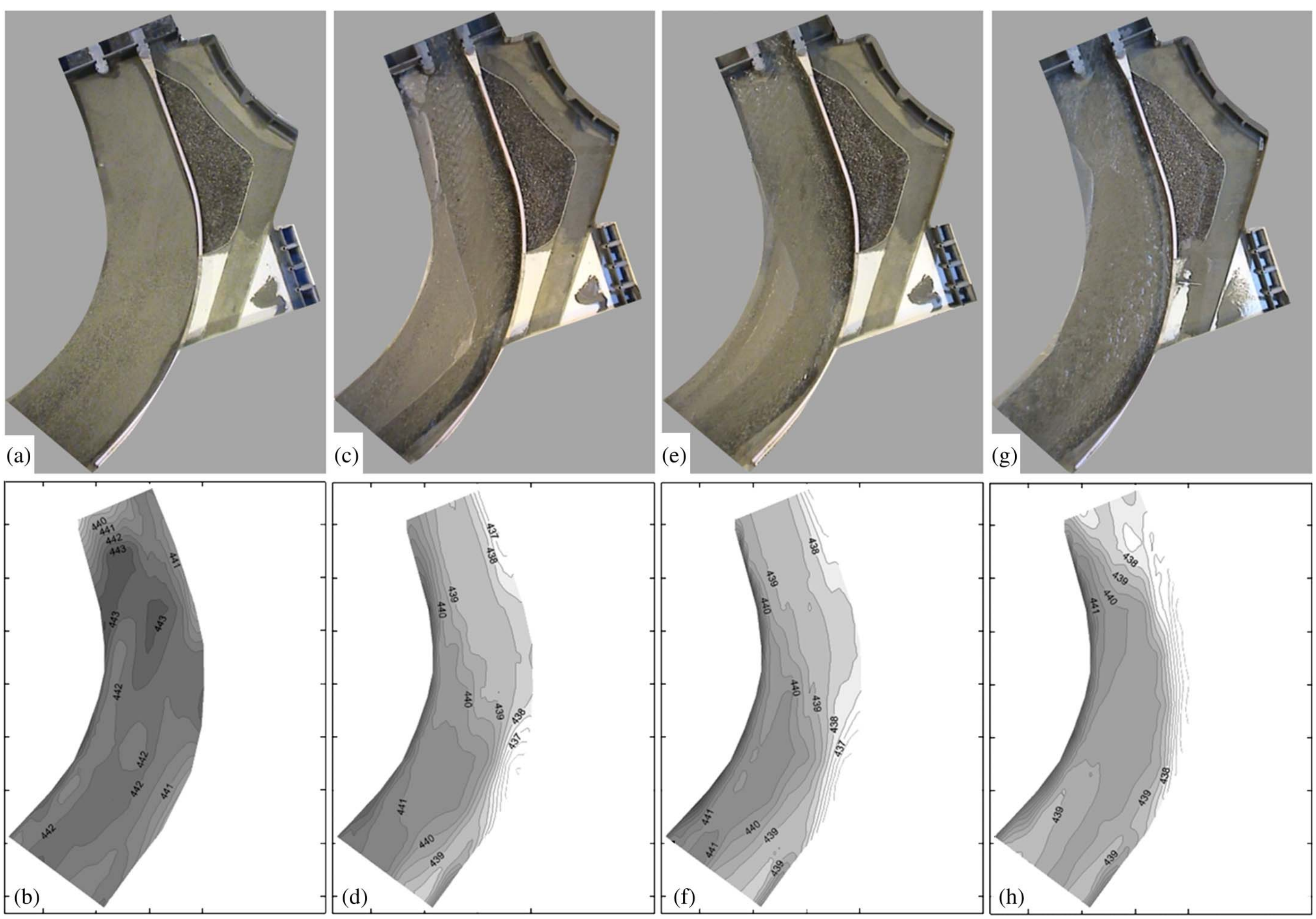

Fig. 9. Deposits and bed elevation for Lavey+ configuration: (a, b) before flushing; (c, d) after flushing with $Q_F = 100\ \mathrm{m}^3/\mathrm{s}$; (e, f) with $Q_F = 150\ \mathrm{m}^3/\mathrm{s}$; (g, h) with $Q_F = 200\ \mathrm{m}^3/\mathrm{s}$

the flow condition that minimizes water losses for a certain sediment flushing volume, the flushing sequences were started with the Lavey+ reference bathymetry for three different discharges (Q_R at 100, 150, and 200 m^3/s).

For the highest discharge, when Q_R is 200 m^3/s, slightly more than half ($V_F = 30{,}000$ m^3) of the initial sediment volume (V_0) in the retention zone was evacuated during the flushing period t_F, whereas only 37% ($V_F = 21{,}000$ m^3) could be flushed when Q_R is 100 m^3/s (Fig. 10). Flushing when Q_R is 150 m^3/s allowed the evacuation of 44% ($V_F = 25{,}000$ m^3) of the initial deposits (V_0). Compared with the cumulated flushed sediment volume (V_F) for the existing hydropower scheme, two to three times the quantity of sediments could be removed from the reservoir. Fig. 9 shows the bathymetry before and after flushing for when Q_R is 100, 150, and 200 m^3/s.

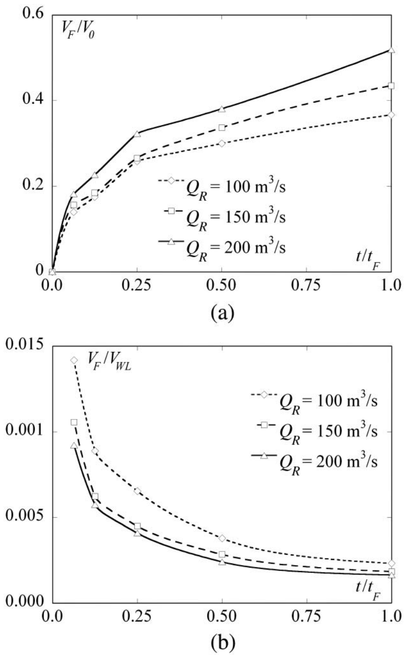

Fig. 10. Flushing behavior of the Lavey+ configuration for three different discharges Q_R with: (a) the relative flushing performance (V_F/V_0); (b) the relative flushing efficiency (V_F/V_{WL}), as a function of normalized flushing time (t/t_F); V_F = cumulated flushed sediment volume; V_0 = volume deposited before flushing; V_{WL} = cumulated volume of water losses; t = cumulated flushing time; and t_F = total flushing time (24.0 h)

Approximately 60 to 70% of V_F could be flushed through the weir during the first quarter of the total flushing period (6 h in this case). The high initial performance results from the erosion of deposits near the weir, where sediments are first affected by the high velocities owing to the steep frontal slope phenomenon; this was observed for all flushing discharges. From this initiation time until $t/t_F = 0.25$, the slope became flatter and therefore the gradients of the two ratio curves, as represented in Fig. 10, decreased considerably. This indicates that flushing efficiency diminishes with time. The period between $t/t_F = 0.25$ and 1 was characterized by continuous sediment transit from the backwaters to the weir, and by the progressive lowering of the river bed in the retention zone. At the beginning of the flushing procedure, the efficiency reached maximum values of nearly 1.5% for low flushing discharge, whereas toward the end a V_F/V_{WL} rate of 0.25% was reached independently of the flushing discharge.

Fig. 11 presents the river bed evolution along the retention zone as a function of time for a flushing discharge at which Q_F is 150 m^3/s. The V_{cum}/V_0 ratio, which is the remaining cumulated volume (V_{cum}) divided by the initial volume (V_0), is plotted as a function of the dimensionless distance (x/B) to the dam in which B is the river weir width of 48 m and x the coordinate on the river axis, starting at the weir and proceeding upstream.

As Fig. 11 shows, significant erosion occurred during the first 1.5 h of flushing. More than 60% of the sediments were flushed within the first 6 h. During the experiments, a long flushing led to a gradual decrease in the bed elevation in the upstream portion of the modeled reservoir reach.

For short flushing, it is therefore better to apply low discharges because erosion is more efficient when water losses (and therefore energy losses) are minimized. High flushing discharges must be available to limit flushing durations.

High discharges that exceed turbine capacity could sluice deposits located immediately upstream of the dam through gates 1 and 2. Five scenarios with different sluicing discharges were tested on the hydraulic model. Sluicing performance was again determined for 24 h by analyzing the V_S/V_0 ratio, which is the cumulated sluiced sediment volume (V_S) divided by the initial deposited volume before operation (V_0). Sluicing discharges (Q_S) had to reach 245 m^3/s to initiate significant erosion at constant operation level in the reservoir. In this case, a sluicing performance in which V_S/V_0 is 6.6% could only be achieved, revealing that sluicing operations perform less well (by approximately eight times) than flushing by lowering the reservoir.

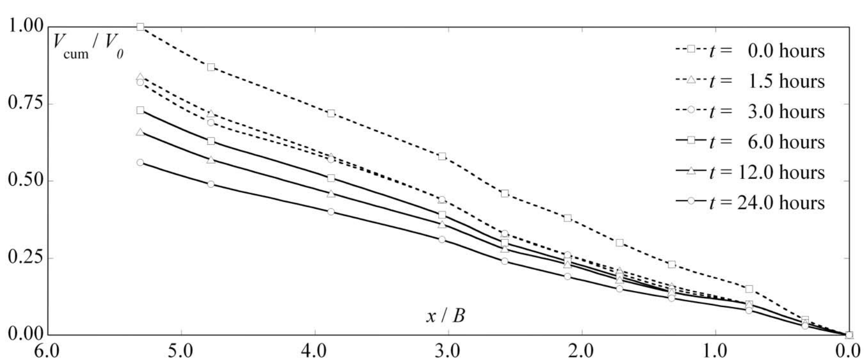

Fig. 11. Evolution of relative sediment volume (V_{cum}/V_0) in the investigation perimeter of the Lavey+ configuration during a flushing operation with a discharge (Q_F) of 1,500 m^3/s, as a function of dimensionless distance to the dam x/B; V_{cum} = remaining cumulated sediment volume; V_0 = volume deposited before flushing; x = coordinate on the river axis ($x = 0$ at the dam); and B = river weir width (48 m)

Conclusions

Reservoir sedimentation resulting from bed and suspended load, endangers the safe and economic operation of the Lavey run-of-river HPP. The optimal sediment management strategy contains temporal sediment storage in the reservoir with bed elevation monitoring for flushing initialization. For economic reasons (i.e., water and energy losses) and ecologic reasons (i.e., effect on downstream habitat), flushing operations should be as short and infrequent as possible. A flushing scenario with maximum efficiency could be identified by physical modeling tests. Data obtained from sedimentation and flushing monitoring with prototype data validated the hydraulic model.

The design and operation optimization process for the enhancement project Lavey+ allowed the following main conclusions:

- The progressively constricting flushing channel at the inner bend acts as a sediment trap and avoids sediment entrainment toward the water intakes,
- Alternately opening gate 1 on the left bank and center gate 2 during flushing increases its efficiency,
- Starting with the center gate (i.e., gate 2 in this case) gives better results,
- Flushing performance, V_F/V_0, is between 37 and 51%, depending on the discharge,
- After the initial flushing of the deposits close to the weir, sediments are transported from the backwaters through the retention zone to the weir,
- More than 60% of flushed sediments are evacuated during the first 6 h,
- Low discharges induce low water losses and therefore high efficiency V_F/V_{WL}, and
- Sluicing performance is less than 7%, even for high discharges.

The training wall in front of the water intakes protects the intakes against sediment entrainment without negatively influencing flow patterns during operation and flood mode. As soon as the bed elevation reaches El. 443, sediments start to cross the notch of the training wall at its downstream end and settle down in front of the new water intake. Continuous monitoring of bed evolution is recommended at three locations in the retention zone at the downstream end of the notch: two sensors on both sides of the training wall and one sensor on the inner river bank. Such prototype monitoring allows recognition of the beginning of sediment entrainment and definition of the next flushing operation with the most suitable conditions (i.e., bed form and elevation, discharge, gate openings). The inner and outer sides of the reservoir can be independently flushed. During floods, sediments are primarily eroded along the wall, thereby forming a channel with concentrated flow at the outer bank. Flushing performance would be inefficient after such events, and therefore refilling of the scoured channel along the training wall has to be attempted. Discharges exceeding turbine capacity should be sluiced by the gates to delay the flushing operation.

The new power tunnel with the additional water intake of the enhanced Lavey+ HPP reduces head losses and therefore increases energy production. The new design of the reservoir allows short and efficient flushing operations, which are not possible for the current scheme. Optimized sediment management reduces inundation risk at the Lavey dam site and reduces sediment entrainment into the power tunnels. In this case, boundary conditions concerning reservoir geometry, sediment load, and discharge range were known. Therefore, general parameter variation was unnecessary. The proposed design and operation principles can nevertheless be helpful for similar cases of run-of-river HPPs.

Acknowledgments

The study was funded by the City of Lausanne, represented by the Services Industriels (SIL), and followed by the consulting engineers Stucky SA, Hydrocosmos SA, and Bonnard & Gardel SA.

Notation

The following symbols are used in this paper:

B = river width at the Lavey dam (48 m);
d_i = particle diameter of fraction i;
d_m = mean particle diameter;
d_{30} = 30% fraction particle diameter;
d_{90} = 90% fraction particle diameter;
Q_a = mean annual discharge (180 m^3/s);
Q_R = discharge of Rhone River upstream of Lavey;
Q_{TE} = turbining discharge of existing water intake;
Q_{TN} = turbining discharge of new water intake;
R^* = boundary Reynolds number;
t = cumulated time;
t_F = total flushing duration (24.0 h);
U_* = shear velocity;
V_0 = volume deposited before flushing;
V_{cum} = remaining cumulated sediment volume;
V_F = cumulated flushed sediment volume;
V_S = cumulated sluiced sediment volume;
V_{WL} = cumulated volume of water losses;
x = river coordinate along axis in upstream direction, ($x = 0$ at Lavey weir);
γ = specific weight of water (1,000 N/m^3)
γ_s = specific weight of sediment (2,650 N/m^3);
τ_0 = shear stress;
τ_* = dimensionless shear stress; and
ν = cinematic viscosity of water (10^{-6} m^2/s).

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