Sediment Management in the Solis Reservoir Using a Bypass Tunnel

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Abstract

The Solis reservoir is located in the Alps in Grisons, Switzerland and is operated by the electric power company of Zurich (ewz). Since its construction in 1986, high sediment input during flood events has led to major aggradations in the reservoir. Up to date, nearly half of the original reservoir volume has been filled with sediments from upstream mountain torrents. The deltaic deposition starts extending into the active water volume. Therefore, ewz plans a sediment bypass tunnel to flush the incoming bedload around the dam to the downstream reach. In a first step the reservoir level during flood events is lowered to the minimum operation level. The delta is subjected to free surface flow and the bedload is transported over the delta and deposited further downstream. This sediment relocation decreases the delta volume within the active storage. During further flood events, the incoming sediment is led to the bypass tunnel intake using a guiding structure and flushed through the tunnel. If the flood exceeds the capacity of the bypass tunnel, the surplus flow passes the tunnel intake towards the bottom outlets with the bedload still being flushed through the tunnel. A skimming wall located upstream from the tunnel intake prevents driftwood blocking by leading it to the reservoir front where it can be safely removed. Both the sediment relocation due to water level drawdown and the flushing through the bypass tunnel are investigated and optimized in a hydraulic model at the Laboratory of Hydraulics, Hydrology and Glaciology (VAW) of ETH Zurich. Additionally, the sediment relocation process in the model is compared with a relocation test in the prototype.

Introduction

The Solis reservoir is located in the Swiss Alps and is operated by the electric power company of Zurich (ewz). The dam, built in 1986, is located downstream of Tiefencastel in the canton of Grisons. It is a 61 m high arch dam with a crest length of 75 m. The dam retains the water from the torrent Albula, resulting in a 3 km long and narrow reservoir which supplies the two hydropower stations Rothenbrunnen and Sils. The original reservoir capacity was about 4.1 million m^3. The annual inflow of about 517 million m^3 coming from a catchment area of 900 km^2 is large compared to the reservoir volume, resulting in a capacity inflow ratio of 0.008. The inflow to the reservoir is low in winter due to snowfall. Flood events occur during summer time caused by both snow melt and rainfall. The hundred year flood event is about 280 m^3/s. The average sediment input into the reservoir is approximately 110´000 m^3 per year, depending on the intensity of the flood events. About 30´000 m^3/a of sediment are removed by a gravel plant located at the reservoir inlet.

During the complete operation period of the reservoir since 1986, the reservoir level was kept between the minimum operation level of 816.00 m a.s.l and the maximum operation level of 823.75 m a.s.l, according to the active volume range. Especially during flood periods with high discharge and high sediment input rates the water level was kept on the maximum operation level. Thereby the delta formation of coarse material was forced as the foreset bed progressed towards the dam. The topset bed was lifted up simultaneously (Figure 1). The sediment aggradation began to rise over the minimum operation level, decreasing the active volume.

The delta has not reached the outlet structures yet. However, suspended sediments settled near the bottom outlets. Thus the dam operator faces three problems due to aggradation: (1) Risk of sediment blocking the outlet structures; (2) Reduction of the active volume; and (3) Risk of increase of suspended sediment concentration in the turbine flow. A blocking of the bottom outlets would endanger the entire dam safety. Active storage volume reduction decreases the electricity production and together with increased hydroabrasive wear of the turbines causes financial losses.

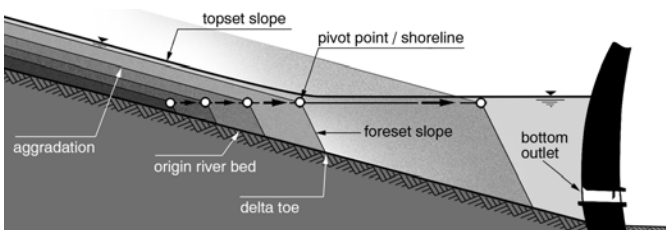

Figure 1: Schematic depiction of an aggradating coarse delta in a reservoir. In case of constant water level the pivot point moves ahead on a horizontal line whereas the topset slope is lifted up parallel to the origin river bed.

To guarantee an economical operation in the future measures against reservoir sedimentation are currently taken. They are separated into two main steps:

- Sediment relocation from the active volume to the dead volume downstream of the foreset;
- Conveyance or removal of incoming sediments through/from the reservoir.

The first step can be achieved as follows. During flood events the water level is lowered, the delta is subjected to free surface flow. The bedload is transported over the delta to the reservoir front and deposited in the remaining dead storage zone. Thus, more active volume for economical operation is provided. This step has already been performed in the prototype and will be compared with the results of the hydraulic model tests below. To comply with the second step, different options for sediment removal or conveyance of bedload have been considered by ewz. They are summarized in the following.

Feasibility study

Many different options are known to decrease aggradation, convey sediments, or remove sediments from the reservoir as shown e.g. in [1], [2], [3], [4] [5], [6] and [7]. The operating company ewz elaborated four different solutions in the run-up phase of the VAW model tests [8], [9]. The main goal of all options is to keep a constant aggradation grade of bedload material in the reservoir. The four options are the following:

1. Sediment dredging;
2. Emptying the reservoir and sluicing the sediments through the bottom outlets in free surface conditions;
3. Lowering the water level below the minimum operation level and flushing the sediments through the bottom outlets in pressurized flow conditions;
4. Lowering the water level to the minimum operation level and flushing the sediments through a bypass tunnel.

While solution one is generally possible, it is inappropriate considering both ecological and economic reasons. The second solution causes various problems. Sluicing sediments through bottom outlets in free surface flow is not effective in this case, because of the low capacity of the existing outlet structures. Another problem is the financial loss due to a complete reservoir drawdown and the resulting plant shutdown.

The third and fourth options have been examined in a hydraulic model test at VAW [8], [10]. In case of flood events the water level of the reservoir is lowered to the minimum operation level, the delta is subjected to free surface flow. The bedload is transported over the delta to the bottom outlets or the bypass tunnel, respectively.

The results show that flushing of sediments through the bottom outlets is possible [8], [9]. However, this option was dropped by ewz due to dam safety risks such as blockage of the outlets. For this reason, ewz commissioned the investigation of flushing the sediments through a bypass tunnel. The main tasks were: (1) Bypassing of all sediments; (2) Optimisation of approach flow conditions towards the bypass tunnel intake; (3) Hydraulic design of the tunnel intake including the tainter gate; and (4) Driftwood blocking tests.

Experimental setup

All tests were carried out in a small scale hydraulic model at VAW [8], [10]. The relevant reservoir section was rebuilt with a scaling factor λ of 45, and is shown in Figure 2. The hydraulic model represented a 1200 m long reservoir section including the arch dam, the bottom outlets and 300 m downstream of the dam. The reservoir has an average width of 80 m.

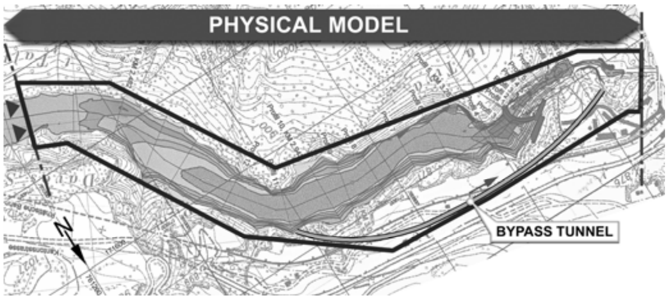

Figure 2: Hydraulic model perimeter. The flow direction is from left to right.

Bedload transport is a free-surface process dominated by gravity, therefore Froude similitude was applied. To respect the Froude (subscript F) similitude, $\lambda_F = 1$ is required, where λ_F is the scale ratio of the Froude number $F = V/(gL)^{0.5}$, with V = flow velocity, g = gravitational acceleration, and L = scaling length. The geometric scale ratio is given by $\lambda = L_p/L_m$, where subscripts p and m refer to prototype and model, respectively. The scale ratios for velocity, time and discharge follow from the Froude similitude as $\lambda_V = \lambda^{0.5}$, $\lambda_t = \lambda^{0.5}$, $\lambda_Q = \lambda^{2.5}$, respectively. Incoming and outgoing flows were measured by magnetic flow meters and the reservoir level was measured by ultrasound. After a model test, the bed morphology was scanned with the help of a laser to obtain a 3D scan of the topography. Bedload, downscaled by means of a method described in [11], was added by a dosing machine in a range from 1 to 35 g/s (model scale) depending on the inflow conditions. The arithmetic mean diameter of the sediment in prototype dimensions is $d_m = 6$ cm. Suspended sediments have not been modelled. Figure 3 shows an overview of the hydraulic model including all measuring instruments.

The tunnel intake was modelled in detail including the first 120 m of the bypass tunnel (Figure 4). The 900 m long tunnel, starting 450 m upstream of the dam on the right bank

leads to a narrow canyon downstream of the reservoir (Figure 2).
All model tests were carried out as follows. The reservoir was first lowered to a certain level. Then the flood discharge was run according to a specific flood event scenario. The water level in the reservoir was kept constant during the entire test. The bedload flushed trough the bypass tunnel was collected in a basket and continuously weighed. After the test run, the reservoir water level was lowered and the sediment delta scanned.

Figure 3: Hydraulic model with (1) sediment dosing machine, (2) bypass tunnel intake, (3) guiding structure, (4) bypass tunnel, (5) laser, (6) intake structure, (7) ultrasound device, (8) spillway, (9) dam and (10) bottom outlets.

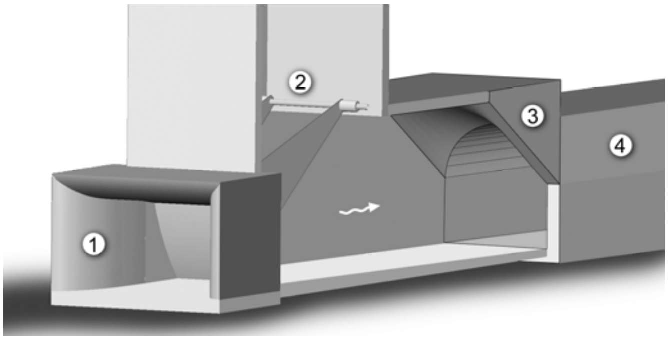

Figure 4: Bypass tunnel intake with (1) inlet trumpet, (2) tainter gate chamber, (3) shape distortion, (4) archway tunnel section.

Model tests and results

Delta development due to water level drawdown
During a flood event in June 2007 with a peak discharge of 100 m^3/s (return period of about 1 to 2 years) the prototype reservoir level was lowered by 11.50 m to a level of 812 m a.s.l. and kept on that level during 8 hours. The water level laid about 5 m below the delta pivot point (Figure 1). Due to the subsequent delta exposition, initial erosion started at the delta front. The vertical and lateral erosion processes advanced upstream over a distance of about 1300 m (Figure 5). Over a range of 700 m upstream of the origin pivot point, the river bed was eroded over its entire width of up to 60 m. Further upstream, the erosion spread over 30 to 70% of the total valley width only. There, the eroded small creeks were 20 to 30 m wide and 1 to 2 m deep. The eroded sediment was deposited further downstream in the reservoir and formed a new, lower delta. Thereby the delta moved by about 94 m. The course of the longitudinal river bed profile was of hyperbolic shape. Based on the pivot point the slope of the longitudinal profile decreased further upstream and adapted the origin slope after 1300 m. Classifying the eroded section into three typical segments, these segments differed in slope and width as follows: in the downstream direction the slope changed from 3‰ over 4‰ to 9‰ and the channel width from 20 m over 30 m to 60 m. The total relocated sediment volume was about 85´000 m^3.
In the hydraulic model three systematic tests were run. Their initial situation was identical: a flat embedded topset bed with a slope of 3‰ and a following foreset slope with an angle of 30°. No morphological patterns were modelled. The simulated 80 hour flood event was subdivided in a first phase of 12 hours duration with a discharge of 80 m^3/s (representing a yearly flood event) and a second phase of 68 hours duration with a discharge of 50 m^3/s (representing a snow-melt runoff). The three tests varied only in their water level drawdown of 2, 4 or 6 m below the pivot point level. All tests showed the same qualitative results described below.

Due to water level drawdown the delta front moved downstream and the new pivot point was located about 1 m below the water level. The lateral propagation of the regressive erosion process was strong near the pivot point and decreased upwards. The speed of the regressive erosion decreased with time due to armoured layered river bed and stopped definitively when the discharge was reduced from 80 to 50 m^3/s. Henceforward, without any increase of the discharge no further erosion occurred. At the end of the test the longitudinal profile was of hyperbolic shape in analogy to the prototype and its slope changed in the downstream direction from 1% over 1.6% to 2.9% (Figure 6). The distance of the foreset to the dam varied in the model test and the prototype (compare Figure 5 and 6). The foreset in the prototype was still 700 m away from the dam whereas the topset bed and the foreset in the hydraulic model were built in the vicinity of the dam to investigate the flushing through the bottom outlets (see above).

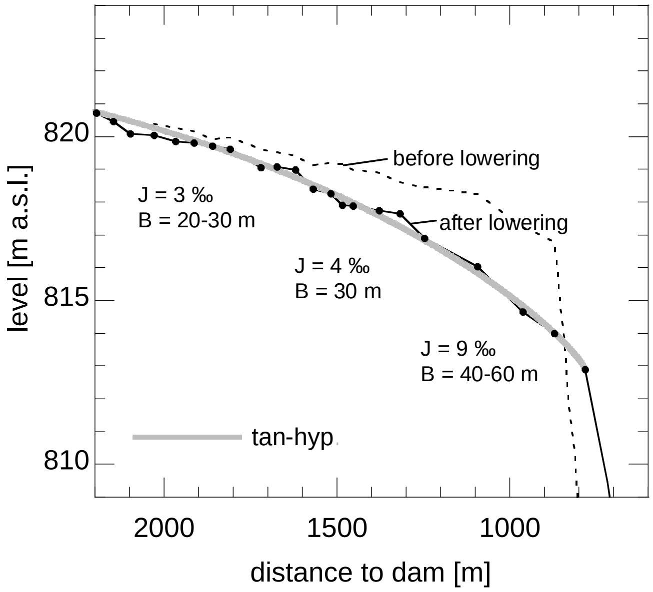

Figure 5: Longitudinal profile before and after the lowering test in prototype. The bed slope and the eroded channel width are denoted for different segments. The end profile follows a tangent hyperbolic function.

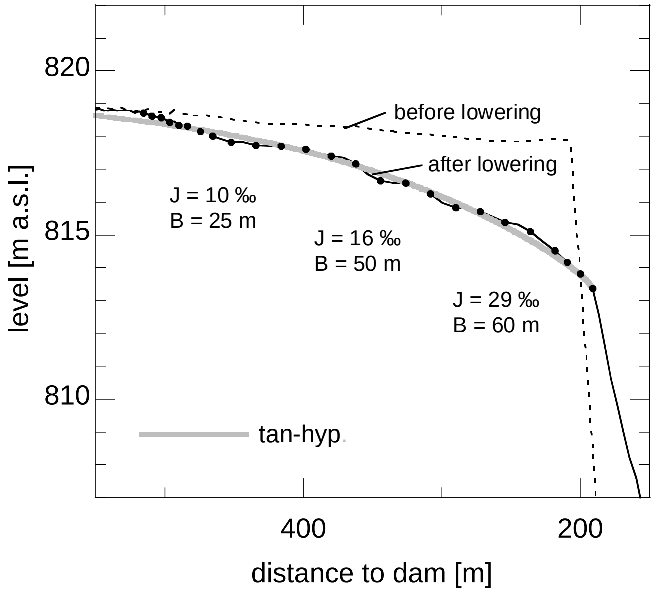

Figure 6: Longitudinal profile before and after the lowering test in the hydraulic model. The bed slope and the eroded channel width are denoted for different segments. The end profile follows a tangent hyperbolic function.

Figure 7 shows the accumulated volume of relocated material during the three tests at four different times. The relocated sediment volume increases significantly with increasing water level drawdown. The larger the drawdown, the bigger the effect of the second discharge sequence on the relocation volume. The topset slope was still large enough after the first discharge sequence to maintain the erosion process even though the discharge had decreased from 80 to 50 m^3/s. The maximum relocated sediment volume in the model test was 71´000 m^3.
Although the relocated sediment volume in the model test is similar to the prototype value, their results are not comparable quantitatively. Whereas the model tests were conducted with a relatively coarse sediment grading curve, the relocated prototype sediments consisted partly of finer material. This fine material could not be modelled in the hydraulic tests. Therefore, the difference of material type leads to higher bed slopes in the model and to a longer erosion distance in the rearward part of the reservoir in prototype. However, the qualitative results of the hydraulic model indicate a proper simulation of the relocation processes and account for the variation of basic parameters like discharge, water level or flood duration. Additionally, the quantitative results of the lowering test in prototype quote the effective erosion potential for natural conditions.

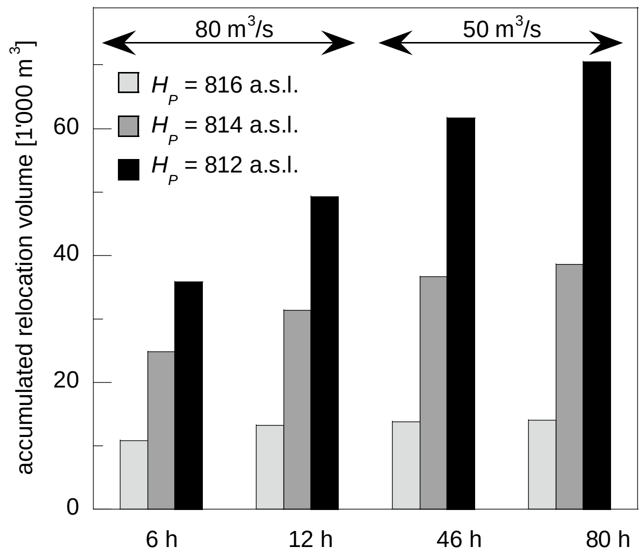

Figure 7: Accumulated relocation volume of sediment in the physical model at different times and for different lowered water levels H_P. The origin of the pivot point of the delta is at 818 m a.s.l. (Prototype dimensions).

Bedload flushing through bypass tunnel

The main challenge of option 4 mentioned above, i.e. flushing the bedload through a bypass tunnel, is the reliable operation of the tunnel up to a hundred year flood event (HQ_{100} = 280 m^3/s). Because the design capacity of the tunnel is limited to 170 m^3/s, a remaining flow of 110 m^3/s then has to pass the tunnel intake towards the bottom outlets. Despite this partial flow to the reservoir front, all incoming bedload still has to be captured by the bypass tunnel. This is achieved by a non-submerged guiding structure placed upstream of the bypass tunnel intake in the reservoir itself. The guiding structure concentrates both the flow and the transported bedload towards the bypass tunnel intake. To maintain a partial flow towards the front of the reservoir, the guiding structure is lowered next to the bypass tunnel intake resulting in an opening (Figure 8).

All model tests were conducted under steady conditions regarding inflow and reservoir level. Tests were conducted with inflows of 80 m^3/s (one year flood), 170 m^3/s (five year flood) and 280 m^3/s (one hundred year flood).

Guiding structure design

An important part of the investigation was the determination of the bedload rate passing the tunnel intake through the opening of the guiding structure towards the reservoir front. The optimisation of the guiding structure and its opening dimensions were therefore of high relevance.

In the first test series the 75 m long guiding structure was situated orthogonal to the stream direction. In most tests the flow upstream of the tunnel intake was along the right reservoir bank towards the tunnel intake so that all bedload was flushed through the tunnel. However, in some tests the flow crossed the reservoir from the right towards the left bank due to changing morphologic formations. The mean flow continued along the guiding structure resulting in scour holes at the guiding structure and bedload input towards the front through the opening. Depending on the flood event, this sediment input could reach up to 15% of the total amount of transported bedload. Discharge lower than the design capacity of the tunnel did not cause any input of bedload or suspended sediment towards the reservoir front as the complete inflow was conducted through the sediment bypass tunnel. A considerable increase of aggradation near the reservoir front due to suspended sediments and bedload only occurs for flood events exceeding the design capacity (HQ_5) of the tunnel.

To concentrate the main flow on the right bank, downstream-facing angled groins were built on the left bank. By means of this measure, scour depths along the guiding structure and bedload input towards the reservoir front decrease. Nevertheless flow can not always be forced towards the right bank by means of groins only. Hence a small rockfill dam is proposed to additionally guide the flow and bedload (Figure 8).

A 6 m high, 140 m long rockfill dam leading from the left bank towards the intake structure was therefore studied in a subsequent test series. The alignment was almost parallel to the flow direction. Hence flow was forced towards the right bank. Input of bedload into the front part of the reservoir was reduced quasi to zero. In case of important floods such as HQ_{100}, input of fine bedload fractions will still be possible due to high turbulence in the vicinity of the tunnel intake. However, input rates should be very low (0 to 4%) according to model test results.

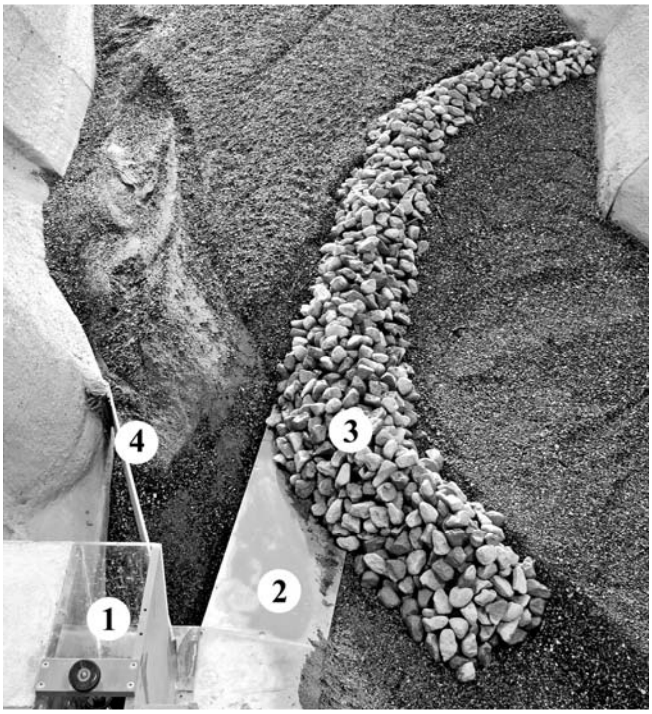

Figure 8: Upstream view from the (1) tunnel intake, (2) opening in the guiding structure and lowered part, respectively, (3) small rock fill dam as guiding structure, and (4) skimming wall.

Bypass tunnel hydraulics

The inflow condition in the tunnel intake is pressurized with an 11.5 m energy head at minimum operation level. Intake side walls and crown are curvilinear, the bottom is plain (Figure 4). A tainter gate regulates the tunnel inflow. The entire sediment bypass tunnel after the tainter gate is then operated under free flow conditions. The design capacity of the bypass tunnel is 170 m^3/s corresponding to a five year flood. A 50 m long, 1% sloped, straight tunnel section begins behind the tainter gate, followed by a curve in plan with a radius of 100 m. The total length of the bypass tunnel is 900 m with a slope of 1.8%, except for the above mentioned inflow section. The tunnel cross section has an archway shape with a width of 4.40 m and a height of 4.68 m.

At the beginning of the model tests, high shock waves in the straight tunnel section occurred for high discharges due to asymmetric inflow conditions caused by the 75 m long guiding structure crossing the reservoir and the topography in the tunnel intake vicinity. These shock waves have to be avoided because of possible change from free-surface to pressurized flow and resulting pulsating flow conditions in the bypass tunnel. Due to the alignment of the guiding structure parallel to the flow direction, symmetric inflow conditions could be attained and shock waves quasi completely avoided.

Driftwood model tests

During flood events, input of driftwood into the reservoir may occur. Up to now driftwood has been transported towards the dam, where flushing over the spillway or removal is possible. Suction of driftwood into the bypass tunnel intake has to be avoided due to a possible blockage. Blockage probability is particularly high for a partially closed tainter gate. If closing of the tainter is not possible due to blocking, the reservoir will be drained off to the bottom level of the intake. Therefore, the possibility of driftwood blocking has to be minimized significantly, e.g. by means of a skimming wall in combination with a partial flow of at least 30 m^3/s that passes by the tunnel intake towards the dam (Figure 9).

A monitoring system to observe the incoming flood events will have to be installed. In case of driftwood transport, the tainter gate has to be lowered to create the partial flow of 30 m^3/s towards the reservoir front. Due to these measures, a great quantity of driftwood is kept away from the tunnel intake. However suction of single logs up to 11 m into the tunnel is still possible. A 100% safety against driftwood blocking is unrealizable.

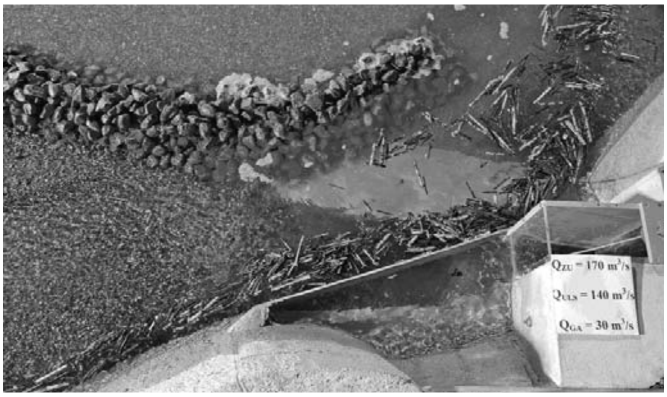

Figure 9: Driftwood model test with flow from left to right. Incoming discharge is 170 m^3/s (HQ_5), 140 m^3/s are conducted through the bypass tunnel, 30 m^3/s pass by the tunnel intake along the skimming wall.

Conclusions

Over about 20 years the Solis reservoir has lost nearly half of its volume due to sedimentation. A first measure against the sedimentation is the relocation of sediments from the active to the dead storage by water level drawdown. The relocation process has been properly reproduced in a hydraulic model at VAW. As a second measure a bypass tunnel in combination with a guiding structure has been designed to convey the incoming bedload from the reservoir to the tail water downstream of the dam. Hydraulic model tests were conducted to analyze the feasibility of this project. The developed measures comply the following requirements:

- Flood events up to a five year return period are completely conveyed through the bypass tunnel in case of no driftwood transport. Hence all incoming sediments are flushed through the bypass tunnel.
- Operation of the bypass tunnel is possible for floods with return periods up to hundred years. The flow exceeding the design capacity passes by the tunnel intake. Nevertheless, due to the guiding structure nearly 100% of the incoming bedload is flushed through the bypass tunnel, independent of the flood event. However, due to the surplus flow, parts of suspended sediments pass by the tunnel intake and settle in the front of the reservoir.
- A skimming wall minimizes the entrainment of driftwood into the bypass tunnel significantly and decreases the risk of blockage. For this purpose, a partial flow of 30 m^3/s has to be maintained towards the dam by lowering the tainter gate.

References

[1] Hotchkiss R.H. (1990). *Reservoir sedimentation and sediment sluicing: experimental and numerical analysis,* Project Report No. 304, St. Anthony Falls Hydraulic Laboratory, University of Minnesota, Minneapolis, Minnesota, USA.

[2] Lai J.-S., Shen H. W. (1996). *Flushing sediment through reservoirs*, Journal of Hydraulic Research, Volume 34, No. 2-1996, pp. 237-255, P.A. Kolkman, ed., Delft, The Netherlands.

[3] Schleiss A., De Cesare G., Jenzer Althaus J. (2010). *Verlandung der Stauseen gefährdet die nachhaltige Nutzung der Wasserkraft.* Wasser Energie Luft 2010, Vol. 1, pp. 31-40 [in German].

[4] Sloff C. J. (1991). *Reservoir sedimentation: a literature survey.* Communications on hydraulic and geotechnical engineering, Report No. 91-2, Delft University of Technology, The Netherlands.

[5] Sumi T., Okano M., Takata Y. (2004). *Reservoir sedimentation management with bypass tunnels in Japan.* Proceedings of the 9th International Symposium on River Sedimentation, Yichang, China, pp. 1036-1043.

[6] Vischer D. (1996). *Internationales Symposium - Verlandung von Stauseen und Stauhaltungen, Sedimentprobleme in Leitungen und Kanälen.* Teil 1 & 2, VAW Mitteilungen 142 & 143, Laboratory of Hydraulics, Hydrology and Glaciology (VAW), D. Vischer, ed., ETH Zurich, Switzerland [in German].

[7] Vischer D., Hager W. H., Casanova C., Joos B., Lier P., Martini O. (1997). *Bypass tunnels to prevent reservoir sedimentation.* Q. 74 R. 37, Proceedings of the 19th Congress of the International Commission on Large Dams (ICOLD), Florence, Italy.

[8] Lais A., Berchtold Th., Mache N. (2008). *Entlandung Stausee Solis – hydraulische Modellversuche zum Variantenstudium ewz*, VAW report 4243, Laboratory of Hydraulics, Hydrology and Glaciology (VAW), H.-E. Minor, ed., ETH Zurich, Switzerland [in German, unpublished].

[9] Oertli C. (2009). *Entlandung des Stausees Solis durch einen Geschiebeumleitstollen.* Wasser Energie Luft, Vol. 1, pp. 5-9 [in German].

[10] Auel C., Berchtold Th., Boes R. (2010). *Entlandung des Stausees Solis mittels Geschiebeumleitstollen.* Wasserwirtschaft, Vol. 4, pp. 117-119 [in German].

[11] Zarn B. (1992). *Lokale Gerinneaufweitung, eine Massnahme zur Sohlenstabilisierung der Emme bei Utzendorf*, VAW Mitteilung 118, Laboratory of Hydraulics, Hydrology and Glaciology (VAW), D. Vischer, ed., ETH Zurich, Switzerland [in German].