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Rehabilitation of the Montsalvens Dam

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**ABSTRACT:** The Montsalvens dam of the “Entreprises Electriques Fribourgeoises” is presently undergoing relevant rehabilitation works involving various innovative design and construction aspects. Completed in 1921, the 52 m high dam may be considered the first modern arch dam built in Europe after the first World War. Although the behaviour of the 75 years old structure is satisfactory, rehabilitation works become necessary to adapt the dam to the actual safety requirements and mitigate the ageing effects. The works include the strengthening of the left dam abutment subject to plastic deformations, the upgrade and rehabilitation of the spillways including the installation of 5.05 m high Hydroplus fuse gates and finally the revision and completion of the monitoring system. The works started on the site in May 1997 and will be completed in October 1998.

1 THE MONTSAVENS SCHEME

1.1 *The power plant*

The Montsalvens scheme situated on the Jogne river, a tributary of the Sarine in the Fribourg Canton, is owned and operated by the “Entreprises Electriques Fribourgeoises (EEF). The construction of the plant started in 1918 to provide peak power to the rapidly growing regional energy market.

The double curvature arch dam closes a 70 m narrow gorge impounding a reservoir of 12.6 mio. m$^3$ of total storage capacity. A 1680 m long headrace tunnel with a flow section of 6.5 m$^2$ supplies the 438 m long penstock. The above ground powerhouse equipped with 5 Francis units of 4.8 MW each generates 62 GWh per year. The gross head of the plant varies between 96 and 122 m with a total nominal discharge of 25 m$^3$/s. Seated into operation in 1921, some of the hydromechanical and electrical components of the plant were successively upgraded and adapted to present standards including namely the replacement in 1987 of the two 1.8 m diameter penstocks by a single 3.0 m diameter pipe.

1.2 *Dam and appurtenant works*

The construction of the 52 m high and 110 m crest length arch dam designed by H.E. Gruner signed the beginning of the development of double curvature arch dams in Europe. The width of the variable radius arches increases at both abutments to optimise the strength distribution in the rock foundations. At crest level (el. 802.30 m a.s.l.), the crown width is 2.0 m increasing to 3.0 m at the abutments and to 22.50 m at the dam base. The cement used for the concrete varied between 250 kg/m$^3$ below el. 765 m a.s.l. and 220 kg/m$^3$ for the upper part with a total placed concrete volume of 26'000 m$^3$. The dam construction was performed in five blocks using hydraulic concrete compacted with pneumatic hampers separated by 1 m wide shear slots. Precast masonry blocks were used as formwork for both the upstream and downstream dam faces.

D.Golliard, R.Bremen and S.Chevalier
The dam foundation consists of thin layers of Malm limestones dipping subvertically with a joint system of variable orientation. During construction works, the left bank excavations disclosed an old preglacial valley nearly parallel to the present riverbed. In addition to a modification of the dam geometry, including the construction of a gravity abutment, the left bank of the dam had to be founded on the rock buttress situated in between the preglacial and the actual valley. The general layout of the dam and the appurtenant structures as configured prior to the rehabilitation works are schematically shown in Figure 1.

![Diagram of dam layout](image)

Figure 1: General layout of the Montsalvans dam and appurtenant structures prior to the rehabilitation works started in May 1997.

With reference to the appurtenant works, the dam is presently equipped with a 57 m³/s capacity bottom outlet located at the dam base and two spillways with a maximum capacity of 75 m³/s each, equipped with the original radial gates installed in 1920. The spillway on the right bank was completed in 1945, following erosion damages occurred in 1944 on the rock buttress below the left bank spillway. The capacity of this spillway was thus reduced by closing one of the two bays and moving the radial gate from the left to the right spillway.

![Image of Montsalvans dam](image)

Figure 2: Montsalvans dam: a) View of the downstream dam face and b) Crown cantilever section including the bottom outlet structure upgraded in 1969.
The water intake structure supplying the headrace tunnel is located on the right bank in between the dam abutment and the right bank spillway. Additional details on the scheme construction and the initial behaviour are given in EEF (1928) and CNSGB (1946). Figure 2a) shows a general view of the downstream dam face, whereas a schematic cross section of the crown cantilever including the bottom outlet is given in Figure 2b).

2 REHABILITATION WORKS

2.1 Strengthening of the left dam abutment

Due to the innovative character of the dam and the particular morphologic conditions of its left abutment an accurate monitoring including a triangulation system and the measurement of the concrete temperatures was installed since the dam construction. In order to improve the monitoring of the plastic deformation of the left abutment toward the valley axis, the triangulation measurements were completed in 1969 with the installation of an inverted plumb line and two rockmeters. Based on the analysis of these measures, it was possible to clearly dissociate the elastic and the plastic deformations of the dam and the left abutment. Since the installation of the inverted plumb line on the left abutment an average plastic deformation of approximately 0.1 mm/year could be measured. In case of relevant variations of the impounding level or complete emptying and filling of the reservoir, the plastic component may attain up to 5mm.

Although the plastic deformations had up to now no effect on the elastic behaviour of the dam, the long term influence of these deformations and its seismic behaviour have been examined in order to ascertain the structural safety of the dam. Figure 3 shows a typical cross section of the rock buttress on the left dam abutment.

The analysis of the abutment stability has been carried out using a two dimensional finite element model taking into account the rock bedding, as schematically shown in Figure 3 and the possible formation of a water table between the rock beds. The forces applied on the rock buttress include furthermore the abutment forces of the dam and seismic loads. After the calibration of the model using the data provided by the monitoring (both elastic and plastic), various load cases were examined in order to identify as accurately as possible the stability conditions of the rock mass.

![Image](image_url)

Figure 3: Typical cross section of the rock buttress on the left dam abutment with a schematic indication of the strengthening works.

D.Golliard, R.Bremen and S.Chevalier
The numerical analyses combined with the monitoring results and the site investigations clearly revealed that the plastic deformations are related to the low shear resistance in between the rock beds. In particular the presence of clay within these layers results in a low shear resistance leading to relatively poor stability conditions in particular in case of seismic loads.

Following a detailed analysis of the shear strength distribution, the actual stability conditions of the dam abutment were not considered satisfactory and design alternatives were examined to strengthen the rock mass. The identified solution consists basically in the increase of the cohesion between the rock layers using passive anchors combined with drainage drillings avoiding the formation of water pressures within the rock.

After an accurate cleaning of the rock surface, 6 m long and 28 mm diameter anchors were drilled in a first step to avoid superficial instabilities of rock blocks during the works. In a second step 170 passive anchors (50 mm diameter) with a length between 15 and 27 m were drilled as schematically shown in Fig.3. Particular care was taken to insure a complete embedding of the anchors with cement grout. The works were completed with the drainage drillings and a 10 cm shotcrete layer covering the whole rock surface as well as the anchors heads. Figure 4 shows the works on the left dam abutment with the nearly 50 m high scaffolding.

![Figure 4: Strengthening works on the left dam abutment during the summer 1997.](image)

2.2 Upgrade and rehabilitation of the spillways

Following various severe floods, the spillway capacities of Swiss dams were revised for most of the old schemes during the last 10 years. This study was equally carried out for the Montsalvens scheme within the dam rehabilitation project. Based on the results of the hydrological analyses combined with need to replace the more than 70 years old spillways radial gates, an upgrade and rehabilitation of both spillways was felt necessary to satisfy the actual flood safety requirements for Swiss dams.

With a newly defined peak discharge of the 1000-years design flood of 346 m³/s, the total spillways capacity had to be significantly increased compared to the actual total capacity of 150 m³/s to avoid any dam overtopping.

The evaluation of various alternatives resulted finally to maintain the actual capacity of the right bank spillway and to increase the capacity of the left one to 280 m³/s. The right bank
spillway will be operated firstly whereas the left one, to be considered as an auxiliary spillway, will be operated only under exceptional flood conditions.

The works on the right bank spillway are thus limited to the replacement of the radial gate with a new plane gate of similar dimensions and to some minor civil works to mitigate the ageing effects of the concrete structures.

As regards the left bank spillway various alternatives were examined to increase the flow capacity and insure satisfactory energy dissipation conditions. In particular the increased capacity should not contribute to a deterioration of the stability conditions of the left dam abutment and avoid any additional erosion of the rock buttress. Based on a comparison of various alternatives taking into account economical, safety and operational criteria a solution including the installation of 4 Hydroplus fuse gates was selected. With a height of 5.05 m, the fuse gates of the Montsalvens dam will be the highest installed in Europe and the first installed in Switzerland for flood control purposes.

The increase of the discharge capacity is obtained by lowering the weir crest, whereas a short chute equipped with a ski jump and a deflector will spread the water jet away from the rock abutment promoting equally the flow aeration and dissipation. Details on characteristics and features of the Hydroplus fuse gates are given in chapter 3. The fabrication of the Hydroplus fuse gates is ongoing whereas the civil works on the spillways are planned during the summer 1998 in order to achieve the spillways upgrade this autumn.

2.3 Monitoring system

Since the installation in 1969 of the inverted plumb line and the rockmeters only minor works were carried out on the monitoring system and devices of the dam. As regards the deformation measurements, the rehabilitation works include the installation of two “Hydro-Quebec” inverted plumb lines, the first 82 m long in the dam crown and the second 66 m in the left abutment. The selection of the “Hydro-Quebec” system is due to the absence of any inspection gallery within the dam or in the abutment. To follow the future deformations of the left abutment, 3 gliding micrometers and 4 rockmeters were installed.

To evaluate the dam behaviour taking into account separately the temperature and the water load, 9 thermometers were installed in the dam body at various levels and depths. Based on the analysis of these temperatures, a deterministic model for the dam monitoring will be implemented in order to rapidly identify any plastic deformation of the dam and of the rock abutment.

The monitoring system is finally completed with 5 piezometers drilled in the alluviums of the pregacial valley. These instruments will be used to quickly identify any abnormal increase of the water table within the rock abutment.

The installation of the additional monitoring devices has been carried out in parallel with the strengthening works on the left abutment during the summer 1997 leaving only to complete the installations for teletransmission.

3. HYDROPLUS FUSE GATES

3.1 Introduction

One of the purposes of the Montsalvens dam rehabilitation, is to increase the discharge capacity of the spillways according to the safety rules and requirements of the Swiss Federal Office of Water Economy. During the design process various alternatives were compared to adapt the discharge capacity of the left auxiliary spillway considering technical, economical and safety aspects.

Following the selection of the Hydroplus alternative, two series of model tests were carried out in the summer 1997 in order to analyse the hydraulic behaviour of the left bank spillway and to verify the efficiency of the fuse gates. Based on the results of these studies the final design of the civil works and of the fuse gates have been performed.

D.Golliard, R.Bremen and S.Chevalier
3.2 Model tests

The first model at geometric scale 1:30 (Froude similarity) was carried out in the Laboratory of Hydraulic Constructions at the Swiss Federal Institute of Technology in Lausanne. The model included part of the reservoir, the left bank spillway with the upstream approach zone, the impact area of the water jet and part of the downstream riverbed. This first model was used to investigate the following main aspects:

- approach conditions to the left bank spillway,
- definition of the optimum shapes of the chute, skijump and deflector in order to limit scouring in the impact area and promote a water jet aeration,
- selection of the optimum tilting sequence of the Hydroplus fuse gates, and
- calibration of the hydraulic characteristics of the spillway including the tilting sequence under normal and exceptional conditions.

Based on the results of these tests, the spillway geometry could be optimised and the most suitable tilting sequence of the fuse gates was selected. It should be mentioned that the tilting sequence was selected taking into account the approach conditions and optimising the shape and position of the water jet on the downstream impact area. Figure 5 shows a general view of the model according to the final spillway geometry.

Figure 5: Model tests of the left bank spillway. General view of the model according to the finally selected spillway geometry. Condition after tilting of the first fuse gate with a prototype discharge of Q=40m³/s.

Following the geometrical definition of the spillway, a second model at a scale 1:10. was built in the chute of the Maigrange scheme owned by EEF. For this model 4 fuse gates 0.505 m high and 0.257 m wide were built according to a preliminary design provided by Hydroplus. The purposes of this second study were to ascertain the operational conditions of the fuse gates and in particular:

- accurate verification of the hydraulic behaviour of the fuse gates and of the tilting sequence,
- evaluation of the stability and tilting characteristics including the definition of the ballast weight in order that each tilting occurs at the required reservoir level,
- analysis of the hydraulic effect of floating debris on the fuse gates, and
- evaluation of the operational reliability of the fuse gates under normal and exceptional conditions including the influence of seal leakage.
Figure 6 shows an upstream and a downstream view of a single fuse gate used for the model tests without the ballast. It should be mentioned that in addition to the hydraulic similarity the gates models were built in order to reproduce at the same scale all the forces acting on the structures in order to correctly simulate their stability and tilting properties.

Figure 6: Fuse gate model (50.5 cm high) used for the second model tests. a) Upstream view and b) Downstream view

3.3 Design of Hydroplus fuse gates

Based on the results of the model studies the design of the Hydroplus fuse gates could be finalised in order to take into account all the hydraulic and geometric constraints. The characteristics of the left bank spillway according to the finally selected design are schematically shown in Figure 7.

Figure 7: General characteristics of the left bank spillway after upgrade and rehabilitation. a) Plan view and b) Cross section. Numbers on fuse gates correspond to tilting sequence.

D.Golliard, R.Bremen and S.Chevalier
In order to achieve the capacity increase, the sill base will be lowered by 0.70 m compared to the present elevation maintaining an unchanged sill width of 10.3 m. For the design of the fuse gates, the relatively high approach velocities to the spillway had to be taken into account in particular after the tilting of the first and the second gate. In order to take into account the hydrodynamic conditions the fuse gates Nr.1 and Nr.2 (see Fig. 7) were designed with standard wells situated at an elevation corresponding to the water profile measured during the model tests.

For fuse gates Nr.3 and Nr.4, a different design had to be selected due to the relatively high approach flow velocities. These gates were equipped with a surge chamber installed on each side of the well supplying the pressure chamber located at the base of the gate. This solution allows to calibrate the tilting of the gate according to the total head (corresponding to the impounding level) and not according to the water surface profile near the gate which may be significantly affected by the approach velocities. This technical modification represents a major innovation of the Hydroplus System. Water levels inside the surge chamber and in the reservoir are almost identical in order that the calibration of the tilting level may be defined independently from the approach conditions. Figure 8 shows an upstream view of the modified Hydroplus fuse gate with at its centre both supply openings of the surge chamber.

Figure 8: Upstream view of the modified Hydroplus fuse gate equipped with a surge chamber. Photograph to be compared with Figure 6a.

REFERENCES

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