Experiences learnt by the rehabilitation of Serra dam (Switzerland)
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Abstract: The Serra dam is located at the southern side of the Simplon Pass, in Canton of Wallis. The old arch dam was constructed in the years 1951–1952 in order to guarantee a daily storage volume for the hydroelectric power plant of Gondo. The plant, owned by the Energie Electrique du Simplon SA (EES), is equipped with three Pelton turbines, with a design flow of 11.5 m³/s and a total capacity of 45.4 MW, providing an average annual production of 177.3 GWh.

The concrete of the dam exhibited signs of expansion due to an alkali-silica reaction which, over the years, led to an irreversible upstream displacement and a heaving of the dam crest. These displacements caused scattered cracks in the dam, especially along the downstream perimeter, which have progressively led to a deterioration of the operating and security conditions of the dam. Following this situation, a rehabilitation of the existing dam was strongly recommended. After analysis of several alternatives, the construction of a new dam few meters downstream of the existing one was decided. The construction works including the demolition of the old dam, started in 2009 and were completed by the end of 2010.

1. Characteristics of the existing dam

1.1 Geometrical features of the dam

The existing Serra dam consists of a vertical cylindrical arch structure with a maximum height of about 20 m. Along a horizontal plane, the dam was constructed by a series of circular arches, with radii varying from 28 m in the central section to 170 m near the banks. The upper arches had a constant thickness of 1.0 m, while the thickness at the foot increased progressively reaching a maximum of about 3.5 m. The dam was constructed in 6 independent blocks approximately 11 m wide, separated by 1 m wide contraction joints. The rather unfavourable design of the dam required a steel reinforcement along the whole downstream face as well as at the upstream foot of the foundation in order to resist tensile stresses (Figure 2). A total concrete volume of 2300 m³ was used for the construction of the dam. The dam had a slender-ness coefficient C (defined by Lombardi as C = S/V/H), where S is the developed surface, V the concrete volume and H the maximum dam height) equal to 40, which is a rather high value compared to other arch dams with similar condition.

1.2 Hydraulic characteristics of the dam

The crest of the dam was located at an elevation of 1260.40 m a.s.l., while the 32.60 m wide spillway at elevation 1278.0 m a.s.l. was located in the dam's central section. The spillway was designed for releasing 175 m³/s at the maximum reservoir elevation (1279.84 m a.s.l.). The dam was equipped with a bottom outlet at the right bank (in front of the headrace tunnel intake) consisting of a circular pipe 1.10 m diameter controlled by a submerged sluice gate installed on the upstream dam face.

During the construction of the dam a diversion tunnel with a capacity of 40 m³/s was excavated on the right bank, by-passing the entire reservoir. This tunnel is presently

Figure 1. Downstream view of the old Serra dam.

Figure 2. Typical cross section of the existing dam.

Figure 3. Plan view of the old Serra dam and appurtenant structures.
used to divert the river during floods and avoid excessive siting of the reservoir.

2. Behaviour of the old dam

2.1 Irreversible displacements of the dam

The displacements of the dam have been checked by means of geodetic measurements since 1952, when it was put into operation. The measuring network was composed of 16 target points located on the downstream face of the dam.

Permanent displacements of the dam began to show in the 1970s. In Figure 4, the total vertical displacements measured from 1952 to 2009 are presented. It could be noticed that the vertical expansion was generally proportional to the height of the blocks with respect to the foundation.

In the central part of the dam, the total displacements exceeded 20 mm. Taking into account the height of a measuring point of 15 m, a displacement of approximately 1.5 mm per meter of concrete could be assumed, which, over a period of 40 years, corresponds to an average annual expansion of about 35 μm/m.

Figure 5 shows the horizontal displacements in radial direction as a function of time for some characteristic points of the dam. The maximum displacement of 65 mm was measured at crest level, while the displacement corresponding to the arch at the height of the spillway attained 50 mm. Such displacements correspond to an average annual expansion of 32 μm/m.

In thermal equivalent terms, the displacement of the dam measured over a period of 40 years may be compared to expansions produced by heating the concrete by approximately 150 °C. In addition, it was noticed that over the last years a progressive acceleration of the displacement in radial direction has occurred (Figure 5). This large irreversible displacement due to concrete expansion led to crack formations spread over the entire dam body. The peripheral crack at the downstream toe of the dam appeared to be of particular importance. It must be noted that this crack is located at the same level of the upstream limit of the rock foundation. Figure 6 shows the probable mechanism leading to the formation of the peripheral crack. Its extension inside the dam could not be determined exactly.

2.2 Hydraulic safety

During an exceptional flood occurred in October 2000, an overflow at the crest was observed. The peak flow rate during this flooding event was evaluated to be 320-340 m³/s, thus much greater than the design flood value of the existing spillway.

Consequently, an update of the hydrological data increased the design flood (1000 years return period) to an estimated value of 540 m³/s. According to Swiss regulations, the dam must be able to evacuate the PMF safely without any danger to its stability. The spillway of the existing dam is therefore not able to assure the required hydraulic safety according to the new hydrological assumptions.

3. Characteristics of the new Serra dam

3.1 General features of the dam

The new dam was constructed adjacent to the existing one as shown in Figure 7. Such an arrangement was mainly defined by topographical conditions. In fact, a deep cliff several tens of meters downstream from the existing dam precludes moving the new dam any further downstream.

From a structural point of view, the new dam can be assimilated to an arch dam with double curvature. Considering the limited dimensions of the structure, it was decided to opt for a simple geometry in which the curva-
ture is defined vertically using a base line with changing direction corresponding to the concreting joints. Horizontally, the arches of the dam are of elliptical shape. The new dam is designed with 6 independent blocks, whose widths vary between 12.0 m and 14.4 m. These blocks are separated by contraction joints, which are grouted with cement slurry after concrete cooling.

The crest of the new dam is designed at 1282.70 m a.s.l., i.e. 1.30 m higher than the old one. The total height of the new dam is 22 m, while the total concrete volume is 3500 m³. The new dam has a rather low slenderness coefficient C equal to 17. However, it is designed to allow the structure to be eventually heightened up to 6 m in order to increase the storage volume of the reservoir.

For the concrete of the new dam non-reactive aggregates were selected in a quarry about 50 km away from the site, in order to further reduce the risk of ASR. The concrete mix design foresee the use of a blast furnace cement CEM III/B 42.5 (dosage of 220 kg/m³).

3.3 Monitoring instrumentation

The monitoring instrumentation is essentially concentrated in the central section of the new dam, where the horizontal displacements are measured using an inverted pendulum installed in the access structure to the valve chamber. The pendulum has two measuring stations; the first at crest level and the second at mid-height (Figure 8). Electric thermometers are installed at three different levels in the same section of the dam.

A piezometer to measure uplift pressure in the dam foundations is also foreseen. The monitoring device is completed by a geodetic measuring network and a levelling at the dam crest.

4. Implementation of the construction works

4.1 Construction program

The construction permit for the new dam was issued in July 2009 by the Cantonal Authority, after the project approval from the Dam Safety Division of the Swiss Federal Office of Energy.

The preparation of job site installations was carried out in August and September 2009. A concrete batching plant was installed on site, whereas the selected non-reactive aggregates for the construction of the new dam were produced in a quarry about 50 km far away and transported on site split in five sieve-fractions. Concrete casting of the main blocks was carried out with a tower crane using a bucket of 1 m³ capacity.

The construction works began in Autumn 2009 with the excavation and casting of block 6 on the left bank. During this period it was necessary to temporary stop the traffic on the cantonal road. The works on site were stopped from the mid-December 2009 to the end of March 2010 due to the presence of snow on site and risk of avalanches along the access road. During the winter suspension, the contractor was able optimize the casting procedures and prepare the adjustable wall formwork in his workshop. In the meantime the contract for the steel lining and the sluice gates of the bottom outlet was signed and its design and construction could start.

In April 2010 the site works began with the excavation of the foundation of the dam, starting from the right banks to the bottom of the valley (Figure 9). The excavation works
4.2 Excavation works of the foundation

The excavation of the foundation at the toe of the existing dam, which was still in operation during works, represented one of the most challenging aspects of the work. Severe vibration thresholds (continuously monitored using geophones installed on the crest of the existing dam) were defined to avoid damage to the still operating dam. In order to limit the vibrations during the excavation phase, vertical cuts, about 10 m long, were made using diamond wire at the upstream and downstream edges of the foundation trenches (Figure 12).

Once this work was completed, the primary excavation was undertaken. Initially, the excavation was to be carried out using explosives. Even if only a minimum quantity of explosives was used, it was not possible to keep the vibrations within the desired limits. Therefore, it was decided to carry out the excavations using a pneumatic hammer mounted on a digger, which allowed adequate limitation of the vibrations however slowed the progress of the excavation work.

A check of the stability of the existing dam revealed insufficient safety coefficients in the zone of the right bank, where carrying out the excavations requires a partial demolition of the downstream toe of the existing dam. It was therefore necessary to place 35 anchor bars Ø 32 mm of length varying from 5 to 8 m in the foundation of the existing dam (Figure 13).

4.3 Behaviour of the existing dam during the construction

During the construction works, and in particular during the excavation phase for the new foundation, a specific monitoring program was adopted in the existing dam. Movements were measured using three inclinometer installations on the dam crest and a temporary pendulum installed in the central section of the spillway. In addition, every two months, geodetic measurements were carried out. These measurements did not reveal any anomalous displacement of the old dam during construction works. In fact, a sound management plan of the reservoir levels during the construction works helped avoiding any excess in displacements.

4.4 Concreting of the dam

The construction of the 6 dam blocks was generally carried out by 3 m high concrete lifts. A total of 34 concrete lifts were necessary to complete the main blocks (without spillway, dam crest and valve chamber). The use of blast furnace cement implicated rather low initial strength of the mass concrete. Therefore a minimum of 4 days were
Figure 12. Excavation of the foundation at the central blocks.

Figure 13. Stabilisation of the downstream toe of the old dam on the right bank.

Figure 14. Concreting of the dam in August 2010.

carried out up to 3 meters depth in rock with a drilling pattern of approximately 2.50 x 2.50 m, resulting in 85 drilling holes. The contact grouting was generally carried out in two phases. At first the lower part of the hole up to a depth of 1.00 m below the foundation was grouted with a maximum pressure of 3 bars. The second phase close to surface between concrete and rock was treated with a maximum pressure of 2 bar. With the exception of a few isolated holes, the grouted volumes (on average 20 litres per meter borehole) were generally very low. The low absorption can be explained by the fact that no explosives were used during the excavation of the foundation, thus reducing the risk of cracking in the rock.

Once the contact grouting was completed, the new grout curtain was then carried out. It consists in 36 boreholes (11 primary, 12 secondary and 13 tertiary) with depth down to 30 m. Primary, secondary and tertiary were drilled separately and then grouted according to the GIN criteria, with a P-V value of 1200 bar/m. Each hole was completely drilled and grouted from the bottom to the top in 3 m long stages by means of an inflatable single packer. Also for the grout curtain the average absorptions (30 litres per meter borehole) can be considered fairly low.

4.5 Grouting works in the foundation

The treatment of the foundation rock, undertaken using cement grout mixes, included two different operations. The first consisted of contact grouting across all the surface of the dam foundation. These grouting works were required before removing the formwork and start the next concrete lift.

Considering the limited height of the dam, every block was completely casted before concreting the adjacent block. Two sets of formworks were used for concreting the 5 blocks in 2010, so that concrete works were only carried out on two blocks simultaneously.

Concrete works in 2010 (30 concrete lifts) required 20 weeks corresponding to an average advance rate of approx. 1.5 concrete lifts per week.

4.6 First filling of the new impound

The procedure for the first filling of the new dam, approved by Dam Safety Division of the Swiss Federal Office of Energy, was scheduled to take place in two stages. During the first day the reservoir was filled to a level of 1274.00 m a.s.l., the equivalent of approximately half the total height of the reservoir. This level was then maintained constant for one week. At the end of this first stage the reservoir level was raised up to the maximum reservoir water level (1278 m a.s.l.) and maintained constant for a further week.

During both filling phases, readings of the monitoring instruments were made at every 2 metres of filling, while during the following phases with constant water level one measurement per day was carried out.

Figure 15 shows the displacements measured at the upper station of the pendulum as well the reservoir level and the temperature of the air during both filling phases and some following weeks with normal operation of the reservoir.

During the first filling phase of 29.11.2010 a downstream displacement of more than 1 mm were measured. This value is a slightly higher than the theoretical elastic deformation of the dam of 0.4 mm, showing that a certain settlement of the dam foundation probably occurred. During the following week with constant reservoir level a further downstream displacement of 0.5 mm were observed (excluding both measurements of 2.12, which do not appear plausible), which can be explained by the drop of the air temperature of about 4 °C.

During the second filling phase, carried out on 7.12, the
downstream displacement measured at the upper station of the pendulum was of about 1 mm as expected. The progressive upstream displacement observed between 7.12 and 13.12 by constant reservoir level was produced by the temperature increase of 12 °C measured from 4.12 to 11.12.

Given the fact that during the first filling operations the behaviour of the dam was in line with the expected displacements, after two weeks the normal operation of reservoir was authorized. The behaviour of the dam during the following weeks is clearly influenced by the fluctuations of temperature, which were exceptionally high in December 2010 and January 2011.

5. Flood management during construction

5.1 Power plant operating procedures during construction

Since the job site is located at the toe if the existing dam, the management of the incoming water was one of the main concern during construction works. Specific operating procedures of the power plant Gondo were implemented in order to protect the job site from flooding during the construction time, whose purposes may be summarized as follows:

- avoid as far as possible any overspill of the old dam;
- enable the workers to evacuate the job site in case of an overspill of the old dam could not be avoided; and
- limit to the minimum the water and consequently the production losses during the construction works.

One important aspect that had to be considered by the definition of the operating procedures is the very small impound volume compared to the catchment area. In fact, during snow melting the 45 MW powerhouse of Gondo should actually be operated as a run-off plant, without any possibility of regulation or even an overspill at the dam. The operating procedure provided the possibility to regulate the incoming flow using the turbines in the powerhouse of Gondo and by operating the two sluice gates of the diversion tunnel (locally operated or remote controlled from the powerhouse). As third measure also the exclusion of the lateral water intake of Lagginbach (only locally controlled and thus requiring a quite long time to be regulated due to the remoteness of the intake) was considered.

It is worth noting that the capacity of the diversion tunnel together with the flow used by the power plant in Gondo, allowed to manage discharges up to about 50 m³/s (corresponding to a flood with a return period of 1 to 2 years) without an overspill of the old dam. In case of larger flood events, a flooding of the job site could not be avoided. In order to create a retention volume in the reservoir and to insure a certain reaction time for implementation of the procedures or for the evacuation of the job site in case of large floods, the maximum operating level was fixed at 1274 m a.s.l. during the construction period. A visual and acoustic alarm signal was also installed at the crown of the old dam in order to warn the workers on site in case the water level rises beyond a certain threshold.

This measure was nevertheless not considered sufficient to prevent any danger of accident on site. A hydrological forecast model was also applied to the catchment area providing an estimation of the expected inflows (see later). Thanks to the measures implemented for controlling the incoming water flows, the engagement of the power plant staff and the favourable weather conditions, the construction works were completed without any interruption or an overspill of the old dam.
No production losses occurred during the construction works with the exception of the 5 weeks needed for the demolition of the old dam. These production losses were estimated to 10 GWh, as estimated in the preliminary studies.

5.2 Hydrological forecast model

In order to increase the security of the civil works, the use of a discharge forecast system provides useful information about the probability of occurrence of a flood during the next days. In this context, a river discharge prediction model has been developed using the RS 3.0 software. The simulation model is based on a conceptual semi-distributed model. The GSM-SOCOMP rainfall-runoff model used in RS 3.0 allows modelling numerous hydrological processes occurring in a complex alpine catchment. The following processes are represented by the model: the spatial distribution of the precipitation and temperature, the snow pack constitution and melt, the soil infiltration as well as the sub-surface and underground base flow, the surface runoff and the glacier melt. In combination with the hydrological processes, the hydropower plants are modelled: the river water intakes with their transfer tunnels, the reservoirs, the spillways and bottom outlets and the hydropower production by the turbines. The model allows then computing of the water transfer in the entire catchment area and following the evolution of the discharge.

This complex information is provided by an Internet platform using the latest geo-mapping facilities. Every type of data could be accessed by a user-friendly spatial interface, such as weather observation and prediction at defined weather stations, discharge at the outlets of the sub-catchments, discharge of each modelled river reach, reservoir level, and hydroelectricity production. If the lastest prediction can be seen, the previous results are still available, in order to help experts to evaluate the probability of a flood occurring during the next 3 days. Finally, the optimal operation of the diversion upstream of the reservoir is automatically provided by the information system, based on the latest inflow forecast and taking into account the actual level of the reservoir.

The discharge forecast is based on three different numerical weather prediction models provided by Meteoswiss, at at 12 h frequency: COSMO2 model for the next 24 h, COSMO7 for the next 72 h and ECMWF for the next 240 h. Precipitation and temperature forecasts are used as input of the hydrological model. This model is then updated by using the discharge gauging stations located in the catchment area. Every hour, a new update of the model and a new discharge forecast is provided on the website.

The difficulty of developing such system is to minimize the risk of false alarms and of missed floods. It is then necessary to communicate the relevant information helping to evaluate the uncertainty of the forecast. Defined animated maps are also provided every hour, which show the last precipitation on the catchment area as well as the predicted precipitations and discharges.

6. Final considerations

Following the expansion of the concrete affected by an ASR, irreversible displacements of the existing arch dam led to a state of cracking spread out over the entire dam. By a further advancement of the concrete expansion the safety of the dam could not be longer assured.

Considering the small dimension of the structure, the reconstruction of a new dam downstream of the existing one was considered the most advantageous solution instead of a rehabilitation of the old structure. This solution allowed equally to limit production losses during the construction works. Moreover, an improvement of the hydraulic safety of the dam by designing a spillway with increased capacity and providing a more efficient bottom outlet was achieved.

The risks, which are accompanied by the construction of the new dam at the downstream toe of the old dam still under operation were easily managed through an appropriate work program, appropriate construction methods (especially during excavation of the foundation) as well as a specific power plant reservoir management schedule. The implementation of a hydrological forecast model demonstrated to be very helpful in order to plan the power plant operations and reduce the risk of flooding of the job site.

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