Safety Enhancement and Strengthening of Les Toules Arch Dam

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Abstract

The paper addresses the case of Les Toules arch dam. In 2003, upgraded seismic regulations and recommendations of the Swiss Federal Office of Energy (FOE), in charge of dam safety in Switzerland required the dam owners to reassess the safety of their structures. In this context, a project was started for safety enhancement and strengthening works at Les Toules arch dam.

Comprehensive studies of the existing dam structure and its behaviour were carried out including static and dynamic analyses. Dynamic analyses revealed the necessity of cantilevers strengthening as well as shear keys in the central part of the dam. The final solution comprises a unique downstream strengthening in the form of abutment thickening, transferring the load from over-loaded cantilevers to the thickened arches, creation of shear keys in the vertical joints, local foundation treatment, and some other, secondary rehabilitation works.

The works started in 2008 and are currently in progress. The contractor in charge of the project has to deal with mountainous conditions, which prevent any works to be done between November and March, because of heavy snowfall and avalanche hazard. The reservoir impounding is scheduled in 2011.

Introduction

Les Toules double curvature arch dam is located in Switzerland close to the southern border with Italy, in Canton of Valais. The Owner of the dam is Forces Motrices du Grand-St-Bernard (FGB). The main **dam characteristics** are outlined in the first section below.

For the strengthening of the arch dam, the design process was performed in two stages: First, the **analysis of the existing dam** was carried out, which allowed calibration of some parameters, thanks to the monitoring data collected during the past decades since beginning of operation in 1964. On this basis, the general dam behaviour could be validated and some specific problems suffered by the dam could be identified and understood. Second, alternatives were studied to select the most technically adapted and economically viable reinforcement solution. The **selected alternative** was then comprehensively designed by means of proper static and dynamic analyses.

Finally some interesting and **specific details of the job site** are described in the last section of the paper.

Dam characteristics

The dam was built in 1960-64 and is actually the heightening of a first, single curvature arch dam built in 1958. The dam enjoys a particular design, with:

- Very slender shape without abutment thickening in a large valley,
- High vertical curvature toward downstream,
- No shear keys,
- An internal so-called pre-pack joint at the contact between the initial dam (1958) and the heightened dam (1964).

The dam is 86 m high and the crest across the valley is 460 m long. The concrete volume is 235'000 m^3.

The dam foundation lies on gneiss and mica schist rocks, forming alternate subvertical strips almost parallel to the valley. The gneiss appears to have better mechanical properties and to be more rigid than the mica schist; thus the dam rests on an alternate series of rock formations which have variable properties and stiffness, especially on the left bank in the vicinity of dam blocs 5 to 7, where small irreversible displacements toward downstream are observed since first reservoir impounding and beginning of operation.

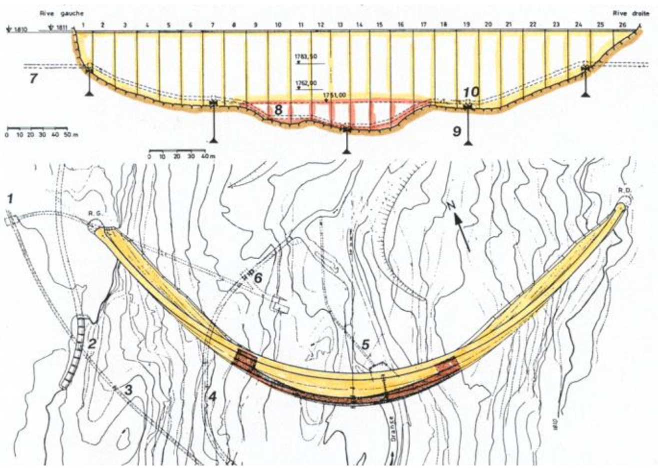

Figure 1: Existing dam upstream face and layout

Studies of existing dam

Seismic design parameters

Les Toules dam was designed in the late 1950s with the knowledge and tools of earthquake engineering of that time. Due to limited knowledge on seismic hazard of the country, most of the old dams in Switzerland have been designed for a peak ground acceleration (PGA) of 0.10g.

As per the recommendations of the Swiss Federal Office of Energy (FOE) [1], the PGA of a dam site should be determined based on the seismic hazard map of the country published in 1977. The map provides seismic intensity isolines for seismic events with return periods of 1’000 and 10’000 years, which is then transformed in PGA.

Based on the dam classification in Switzerland, dams higher than 40 m or whose reservoir volume is greater than 1’000’000 m^3, are considered Class I. Les Toules dam complies with both characteristics, and for a Class I dam, a 10’000-year return period seismic event is demanded as the safety check earthquake. For Les Toules dam, the recommended method gives a PGA of 0.33g which is 3.3 times higher than the original design earthquake of the dam.

To be more specific and relevant, a seismic hazard study was carried out in order to determine a more precise PGA based on local faults and site geology and select three proper recorded earthquakes corresponding with local conditions. The seismic hazard study of the site was performed according to deterministic and probabilistic approaches and resulted in a horizontal PGA of 0.28g and 0.19g for the vertical component.

Calibration of design model

Before launching detailed analyses, calibration of the finite elements model had to be fulfilled for both static and dynamic behaviour of the dam based on concrete test results and measurements.

As for the static calibration, firstly the concrete measured temperature was simulated by means of an analytical transient thermal solution proposed by Stucky-Derron [2], which allowed determining the thermal concrete parameters and boundary conditions. Subsequently, displacements of the model were calibrated on the basis of the data monitored with three plumblines at three different blocs by adjusting the mechanical characteristics of concrete and rock foundation. Some plastic displacements and slight cracking after almost 40 years of operation, made it difficult to obtain a perfect calibration of the finite elements model. Moreover, the behaviour of the pre-pack joint separating both construction phases of dam was studied in two modes: a) joint fully open; b) perfect contact, i.e. monolithic behaviour.

The dynamic calibration of the dam was carried out on the basis of natural frequencies and also mode shapes, both obtained by ambient and forced vibration test of the dam. The measured mode shapes and frequencies were compared with those obtained within the finite elements model for the first ten vibration modes of the dam. The dynamic modulus of the dam concrete and the behaviour of the construction joints could be inferred from the approach and used for the time-history dynamic analysis of the dam.

Analysis of existing dam

Full static and dynamic analyses of the existing dam allowed verifying the safety of the structure based on the FOE recommendations.

The static analysis showed relatively high vertical tensile stresses on the upstream face of the dam for full reservoir load case. These stresses were mostly found at the heel of the cantilevers and on the banks, and reached up to 5 MPa for full reservoir combined winter temperature load case. Such high vertical tensions for static load case could produce cracks on the upstream face of the dam, an opening of the dam/foundation contact, or decompressing of the foundation on the heel of the dam. To some extent, all these phenomena have been observed at les Toules arch dam. The compressive stresses were also significant but acceptable with a maximum value of 12 MPa for static load cases.

The results of the dynamic analysis showed that the dam would experience very high tensile stresses in case of a seismic event. The maximum tensile stress obtained with the final elements model was about 12 MPa in the same zone as the tensions occur due to static load cases. Such high vertical tensions confirmed the necessity of the dam strengthening. In addition, high horizontal tensions were obtained in the central part of the upper arches, which could trigger significant opening of the radial joints. These joints enjoy a helical geometry but without any shear keys, which is unfavorable to withstand large joint opening and therefore, the dynamic stability of the upper parts of the cantilevers was of concern and had to be considered in the strengthening concept.

Selected strengthening alternative

Strengthening alternative study

To remedy the different critical items as identified in the analysis of the existing dam, many solutions were envisaged and investigated in the framework of an extensive alternative study:

- Thickening of the dam section (upstream, downstream, thrust blocks)
- Buttresses
- Implementation of a seismic belt
- Use of prestressed anchors.

Even solutions with no interference with the dam were considered:

- Lowering the maximal operation level
- Building a new dam downstream of the existing dam.

Finally it was found that reinforcing the dam on its downstream face by addition of two lateral strengthening (abutment thickening) on each bank, together with shear columns in the central portion of the arch was the most optimized solution, in particular since it does not involve any job in the upstream face (inside the reservoir). With that, the power scheme can be operated during the reinforcement works, although under modified operating rules.

Description of selected strengthening solution

Figures 2 and 3 show the downstream face of the dam and two cross-sections of the existing dam together with the reinforcement. The strengthening of the dam is shown with dark grey, whilst the existing dam is shown with hatches.

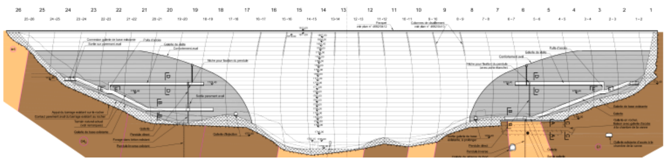

Figure 2: Downstream face of dam

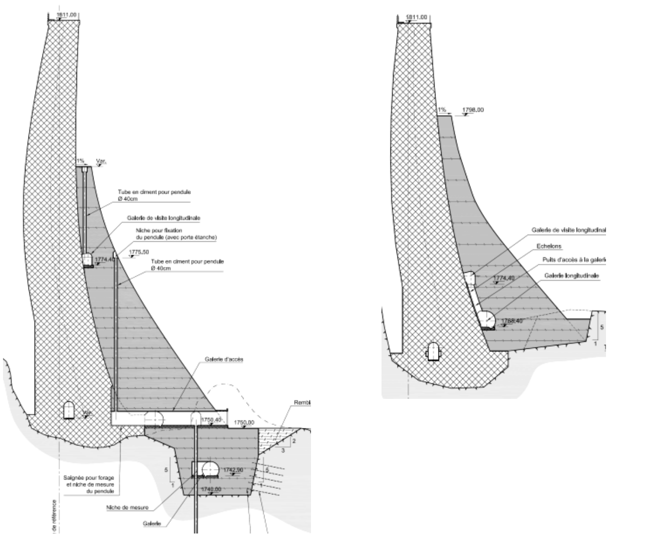

Figure 3: Cross-sections of existing dam and reinforcement; Left: Bloc 6, 10 m local deepening into the foundation; Right: Typical cross-section

To respond to seismic requirements, the thickening of abutments in both the left and right banks allows the transfer of the load from over-loaded cantilevers to thickened arches. Both abutment thickenings are designed with galleries located at the interface of both old and new structures. During the construction, the galleries serve as access and to implement the concrete post-cooling system, dam contraction joint grouting and rock foundation treatment (consolidation grouting + drainage). On completion of the reinforcement works and during operation, the galleries serve mainly to access the monitoring equipment.

Still for seismic purposes and along with these two 30’000 m^3 concrete volumes added on the downstream face of the existing dam, it was found necessary to lock together the central cantilevers, from bloc 8 to bloc 17, so as to prevent the blocs to move independently one from each other in case of earthquake. Nine so-called shear columns were designed for that purpose. To behave properly, they have to be thoroughly aligned and centered in the helical plan of each dam joint and are made of heavily reinforced concrete. The columns are 16 m deep into the existing dam concrete.

Locally in the area of blocs 5, 6 and 7 (left bank), where the weakest geological layer prevails (mica schist rock), the dam abutment stability analysis of the existing dam revealed that the dam foundation was too shallow and not sufficiently deeply founded into the rock mass [3]. To comply with the design requirements, a 10 m deepening of the dam foundation was designed (Figure 3, left), bridging the weak mica schist layer between both stronger, neighbour gneiss layers.

Following the construction of both dam thickenings as shown in Figures 2 and 3, proper foundation treatment including a 15 m deep consolidation grouting and drainage holes is implemented.

In addition to these works, a 1 m high reinforced concrete parapet wall is designed in substitution of the existing steel barrier. This measure allows the increase of the water head by 1 m for the ungated, free-flow spillway located on the left bank, thus substantial increase of the discharge capacity to accommodate the PMF flow condition.

Analysis of strengthened dam

In-depth static and dynamic analyses of the strengthened arch dam were performed, showing that for the static cases, the high tensile stresses observed in the upstream face of the existing dam are reduced by 45% for the critical load case of full reservoir combined with winter temperature. By reinforcing the arches in both banks, the cantilevers are effectively unloaded and the arch effect accordingly increased. The compressive stresses calculated in the strengthened dam remain in the same magnitude as for the existing dam, although for some specific load cases, a stress reduction in the range of 30% is observed.

The dynamic analysis confirmed that the increase of the dam rigidity by thickening of the arches leads to increased values of natural frequencies. A comparison of the results obtained for the existing dam and for the strengthened dam shows a reduction of the tensile stresses by 30%.

Specificities of job site

Water level control

In 2005, according to the results of both static and dynamic analysis of the existing dam, the FOE imposed the Owner to lower the maximum operating level by 10 m at 1800 m.a.s.l. Then, from 2008 and during the whole period of the strengthening works, more stringent operation constraints were decided, imposing the lowering of the reservoir level down to 1780 m.a.s.l., which means 30 m below the maximum operating level. This measure was taken to ensure an increased safety level against floods during construction, release the stress condition in the dam body and foundation, and prevent from any disturbing leakage inflows within the foundations.

In order to keep the reservoir level below 1780 m.a.s.l., several measures can be taken gradually, from a 24 h/day power generation (10 m^3/s outflow), up to a full opening of the bottom outlet gate (70 m^3/s). The ungated, overflow spillway is located on the left bank at an elevation of 1810 m.a.s.l. (crest elevation) and therefore cannot participate in the water outflow from the reservoir.

Quarrying

Taking advantage of the operating constraints during the strengthening works as mentioned above, the quarry is located between 1780 and 1810 m.a.s.l. in the reservoir bed, upstream of the dam at the river mouth, as shown in Figure 4 below. The aggregates and sand necessary to produce the dam concrete are sourced from this quarry.

Figure 4: Quarrying inside the reservoir lowered during construction works

This solution offers several interesting upsides to the project, such as the proximity of the quarry to the dam, thus reduced aggregates and sand conveying distance, as well as a nonexistent environmental impact after impounding, since the quarrying area will be covered by water.

It is to be noted that the existing dam was also built with aggregates and sand extracted from the same quarry. The long term behaviour of the existing dam concrete was checked and found satisfactory; no alkali-aggregates reaction was observed and concrete durability is ensured. Therefore the concrete characteristics and behaviour of the existing dam from the old days to date fully justify the use of site aggregates for the current concrete works.

Construction schedule

The project is expected to last 3.5 years, including winter breaks. The construction schedule is as follows:

2008: Site installations
Local deepening at the left bank and concrete works (blocs 5 to 7)
Drilling/concreting of shear columns at dam crest
Right bank foundation excavation

2009: Right bank concrete works
Drilling/concreting of shear columns at dam crest (end)
Left bank foundation excavation
Upstream parapet concreting

2010: Left bank concrete works
Left and right bank consolidation grouting
Right bank dam contraction joint grouting
Upstream parapet concreting (end)
Partial demobilization

2011: Left bank dam contraction joint grouting
Finishing works
Final demobilization.

Site logistics

The right and left bank abutment thickenings each have a concrete volume of about 30’000 m^3. The specificity of the site in mountainous conditions at 1800 m.a.s.l compels the Contractor to work from April to October. Considering this tight schedule, the decision of the Contractor to work with two 9 hours shifts – a day shift for formwork and preparatory works and a night shift for concrete works – became an evidence.

According to the volumes of the largest concrete lifts, (about 400 m^3), a concrete placement rate of 40m^3/h was required. The layout of the existing dam as well as its particular shape did not allow the installation of a cable crane. Therefore the Contractor opted for a tower crane for the transportation of the concrete to be placed. The distance between both banks forced the Contractor to erect the tower crane at the right bank in 2009, dismantle it at the end of the season and re-erect the crane at the left bank in 2010.

As the batching plant is located on the left bank, trucks were necessary in 2009 for concrete conveying to the right bank, whereas in 2010 for the left bank, only a short conveyor belt was used from the concrete mixer to the bucket at the foot of the tower crane.

Considering that the crane is able to operate between 12 and 15 rotations/h and with respect to a 40 m^3/h concrete placement rate to be ensured, the capacity of the bucket shall be 3 m^3. The load of the full bucket (10 tons), together with the location of the crane and the abutment thickening size lead to a crane capacity of 630 tons meters. The crane is 95 m high and is shown in Figure 5.

Bounding between existing dam and abutment thickening

The bounding between the existing dam and the strengthening was performed by a proper hydrodemolition with high pressure rotating water jets (2500 bars) and with diameter 25 mm steel anchor bars grouted into the existing dam. The density of the steel anchors is 4 pieces/m^2 in a 3 m wide belt in the edge of the contact surface, and 1 piece/m^2 anywhere else. Hydrodemolition was carried out using an automatic mobile gantry as shown in Figure 6 below and allowed an average surface roughness of 2 cm.

Figure 5: General view of the crane and strengthening work in the right bank (2009)

Figure 6: Hydrodemolition as a means to prepare the interface surface between old and new concrete

The contact between the existing dam and the new strengthening is not grouted, to prevent from any risk of unsticking while grouting (jacking effect). Measurements with teledilatometers placed on the interface of both structures showed no relative movements between old and new concrete.

Shear columns

Drilling of the 16 m deep shear columns with a 700 mm diameter required the implementation of a specific, 3 steps methodology. First, 120 mm diameter core drilling was carried out; then boring out with 400 mm diameter; and finally, reaming with a 700 mm diameter, rotative diamond tricone.

Together with the 120 mm core drilling, deviation checking was performed to ensure that the drilled column effectively was aligned with the dam contraction joint. Then, reinforcement was placed and self compacting concrete was poured from the bottom of the hole in an ascending stage.

Figure 7: Placing of reinforcing cage after drilling of 700 mm column and before self compacting concrete pouring

Thermal behaviour of concrete and requirements

The thermo-mechanical analysis showed that the maximal concrete temperature should not exceed 35°C, so as to keep the stresses below the tensile strength of the concrete.

Night concreting, permanent pre-cooling of aggregates with water mist and shady stockpiles, limited concrete lift volumes and installation of post-cooling pipes system each 3 m were the pre- and post-cooling measures implemented in order to comply with the maximal temperature authorized for the concrete works. Figure 8 below illustrates an example of the concrete temperature rise in a 1.5 m high concrete lift. The thermometer was placed at 1.3 m from the base.

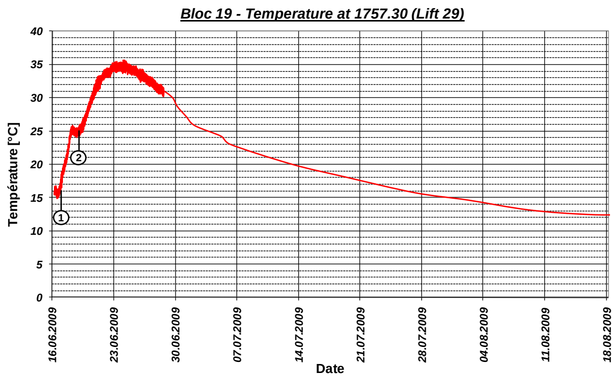

Figure 8: Concrete temperature vs. time curve showing temperature rise caused by cement hydration heat and subsequent effect of post-cooling

Indication 1 on the graph corresponds to the beginning of concrete placement, whereas indication 2 shows the thermal effect of placing the next, upper concrete lift (after 3 days). The average temperature rise reached 22°C, which means that a maximal fresh concrete temperature of 13°C was allowed in order to meet the requirements. During summer, when fresh concrete temperature was too high and caused the exceeding of the maximal authorized peak temperature (35°C), additional cooling pipes were placed, thus reducing to 1.5 m the spacing between two successive post-cooling layers.

Concrete specifications

The static and dynamic analyses showed that the stresses in the abutments thickenings in the vicinity of the existing dam as well as in a 3 m belt on the edge were higher than in other locations.

Therefore, two types of concrete with different cement contents were used for the abutment thickenings. In 2008, extensive concrete investigations were performed to fix the most suitable concrete mixes to comply with the technical requirements. Final mixes are: 190 kg/m^3 of cement and 110 kg/m^3 of fly ash, with 1.2% of plasticizer (concrete type I). For concrete type II, a reduction by 15 kg of cement was balanced with an increase by 15 kg of fly ash. The cement has low hydration heat. The maximal size aggregate is 80 mm. Cubic compressive strength requirements (20 x 20 x 20 cm) were set as shown in the following table.

TABLE 1: COMPRESSIVE STRENGTHS

Mass Concrete	28 days [MPa]	365 days [MPa]
Type I	30	50
Type II	25	37

Routine tests are ensured by the site laboratory installed by the Contractor and samples are taken on a daily basis.

The stringent requirements regarding compressive strength and maximal allowable concrete temperature made the concrete mixes definition a sensitive technical issue. However, satisfactory results have been recorded for the time being.

Conclusion

Due to very specific and local conditions described in the paper, Les Toules double curvature arch dam enjoyed a unusual design in the 1950s and experienced particular behaviour during the past 40 years of operation. However, due to the advent of up-graded seismic directives issued by the FOE, it was found necessary in 2005 to increase the safety of the dam by reinforcing the structure. In-depth studies were performed, first by the back-analysis of the existing dam, which allowed identifying the critical items to be addressed. On this basis, a strengthening project was developed, which mainly considers the construction of two, 30’000 m^3 concrete volume, dam thickenings added onto the downstream face of the existing dam.

The construction started in 2008 and is scheduled to be completed in time in 2011. Common technical items belonging to dam technology have been implemented, but needed to be adapted to the particular site and strengthening work type; among others the hydropower scheme is still being operated during construction, and the job site must cope with rather extreme weather condition.

Figure 9: Aerial view of the dam during strengthening works

Acknowledgements

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References

[1] Swiss Federal Office of Energy (2003). *Sécurité des ouvrages d’accumulation. Documentation de base pour la vérification des ouvrages d’accumulation aux séismes.*

[2] Stucky A., Derron M.-H. (1957). *Problèmes thermiques poses par la construction des barrages-réservoirs.*

[3] Cekerevac C., Wohnlich A. (2009). *Reassessment of dam foundation stability – The case of three Swiss dams.* 2nd International Conference, Long Term Behaviour of Dams, Graz, October 2009, pp. 733-738.