Chambon Dam Reinforcement Works

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Abstract
Chambon, a 137 m high concrete gravity dam completed in 1935, is affected by severe alkali-aggregate reaction, causing several types of pathologies as a result of the concrete expansion. They mainly result in important vertical cracking of the structure, likely to affect its integrity under earthquake, important shear stresses in the structure and deformation of the crest. Reinforcement works were performed from 1991 to 1997 to guarantee the safe operation of the dam: mainly a series of slot cutting, and a drained PVC sealing geomembrane system, installed to provide waterproofing protection at the upstream face. These measures allowed an extension of service of 20 years.
New reinforcement works were performed in 2013-2014, in the continuity of those already performed in the 1990's: (i) installation of 415 prestressed tendons crossing the structure from upstream to downstream, supplemented with a carbon fiber composite net on the upstream face, (ii) realization of 7 vertical diamond wire slot cuts and (iii) replacement of the existing sealing geomembrane to allow the works.
This paper explains the approach that led to the choice of technologies based on the results of investigations and finite elements modeling.

Keywords: AAR, Modeling, Tendons, Cuttings, Membrane.

1. INTRODUCTION

Chambon dam is a 137 m high cyclopean concrete gravity dam, located on the Romanche river in the french Alps at elevation of 1,000 m. The crest length is about 300 m and the dam is curved in the left bank zone. It has been built from 1929 to 1935 and creates a 50 million m^3 reservoir, which supplies a 116 MW power plant operated by EDF since 1946.

Chambon dam suffers Alkali-Aggregate Reaction. This has been discovered in 1958, when unexplained cracks and deformations were noticed. Then, the central section moved 1 mm/year downstream, while the curved left wing moved 5 mm/year upstream. The crest rose 3.6 mm/year. A horizontal crack opened between the drainage gallery of the spillway and the downstream face. A leakage flow increased yearly by 3 l/min in the downstream left bank rock abutment.

Since then, numerous investigations and laboratory tests were implemented. Alkali-Aggregate Reaction has been formally identified in the late 70’s. The aggregates, considered as the main cause, came from the local quarry of gneiss with numerous layers of black micaschists.

2. THE FIRST REINFORCEMENT WORKS CAMPAIGN (1991-1997)

In 1991, a working group gathering the owner EDF, the designer Coyne et Bellier and the Public Authorities proposed an alternative to the construction of a new dam. Since the longitudinal compression stress measured varied from 2 to 6 MPa, the first idea was to separate the spillway from the dam, in view of eliminating the risk of gates blocking or piles shearing. A new underground spillway was built in the left rock abutment to provide full safety during the reinforcement works.

The second aim was to relieve the swelling stresses by innovative slot cutting through the upper part of the dam (Chambon dam was the second dam in the world to experiment this type of works). An isotropic temperature rise-swelling law was implemented in a no-tension model by Coyne et Bellier in an attempt to predict the effect of slot cutting on the dam swelling. The qualitative dam behavior was well described; however, longitudinal compressive stresses were two or three times higher than the measured ones. In such conditions, the stresses calculated by the model were unacceptable after 10 to 30 years at the top of the left bank. The role of the slot cutting was also to relieve such unacceptable stresses. However, the model showed that the effects of this stress relief should not last over 20 years and that other works should be required in the future.

Before cutting, the significant open lift joints spotted in the upper part of the downstream face (5 to 20 m below the crest, 2 to 3 m deep over a 200 m length) were grouted (Figure 1 – a). This process was necessary to ensure static and seismic stability of blocks separated by horizontal lift joints and vertical saw cuts. Also all cracks were filled and sealed with several grouts (0.5 < W/C < 2) at a maximum pressure of 0.2 MPa in two seasons: 1992 and 1993. 20 m^3 of grout were used. Quantities grouted in the curved area were twice those in the central section.

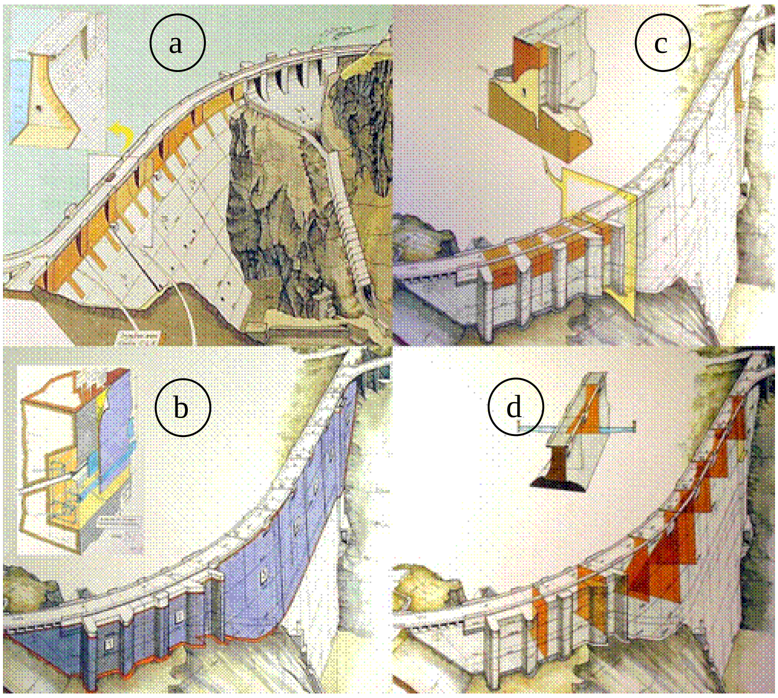

Figure 1. 1990's main works in relation with AAR
a: crack grouting, b: upstream sealing, c: old spillway concrete, d: slot cuts

From 1991 to 1997, 9000 m² of Carpi PVC sealing geomembrane were installed on the top 40 m of the upstream face in order to remove the uplift in the upper part of the dam, due to cracks opening, and to solve the question of the slot watertightness (Figure 1 – b). The upstream face was smoothened with sprayed mortar in advance. A reinforced beam was concreted at the base of the geomembrane. The drainage and puncture protection was ensured by a geogrid. The geomembrane was tightened between vertical profiles 1.85 m apart. The drainage system was divided into 9 isolated compartments with their own leakage monitoring. In such manner the localization of the entry point of any leak was facilitated. Thanks to the complementary foundation grouting works and the concrete shell added at the spillway toe, the left abutment leakage dropped from 50 l/min in 1994 to 22 l/min in 1995 and up to 17 l/min in 1997.

Before starting the slot-cutting works, the initial spillway was decommissioned. The gates were removed and the openings were filled with 4,000 m^3 of concrete (Figure 1 – c).

Three campaigns of slut-cutting were required from 1995 to 1997: 2 slots in 1995, 3 in 1996 and 3 in 1997 were completed with 11 mm diamond wire from 18 to 32 m in depth below the crest (Figure 1 – d). The main effects of slot cuts on the dam were: 1) the return of the curved part toward downstream, and 2) the return of the bank side blocks toward the center of the dam. During the last campaign, the 3 slots did not close and surrounding slots and joints opened, this was the sign of the effectiveness of the stress relief.

3. The New Diagnosis (2007-2010)

Fifteen years after the first reinforcement works, the slots closure monitoring showed a slow crest recompression and the pendulums exhibited the restart of the curved left wing movements toward upstream. Following this movement, the horizontal crack below the spillway reopened.

According to those observations, an extensive investigation campaign was launched between 2007 and 2010. The aim was the diagnosis of the dam condition and the definition of the next works to continue the operation of the dam in safe conditions. These investigations included drill holes in the dam body to identify the extent of crack networks, detailed inspection of the concrete-rock interface by digital borehole logging inside drill holes, identification and characterization of swelling laboratory tests on samples representative of the different areas of the dam, on-site stress measurement with flat jack devices inside the galleries and from the downstream face. Overcoring in-situ tests were also performed in order to get the three dimensional stress tensors, leading to more relevant results of the compressive stresses. Foundation modulus measurements by dilatometer tests were also carried out.

This extensive campaign showed the almost systematic presence of a vertical cracking along the elevation drainage curtain in the upper part of the dam, which led to the conclusion that it was not possible to exclude some upstream concrete blocks instability in earthquake conditions. They might be precut by the elevation drainage curtain and extended horizontal or vertical discontinuities. The presence of these cracks was then explained by several factors: (i) vertical drains of diameter 800 mm (in red and green on Figure 2) and 300 mm (in blue on Figure 2) are separated by an average distance of 3.20 m; they could work as a “precut line” along the axis of the dam, (ii) the differential expansion rate between the upstream concrete containing 250 kg/m^3 of cement and the downstream one containing 150 kg/m^3 could create shear stresses along the same surface (Figure 2), (iii) the geometry of the dam, curved in its left bank side. Most of these cracks are observed in drilling holes through the curved zone and are several millimeters to more than 1 centimeter open. As the different cracks are most of time not linked inside the dam body, the presence of a continuous upstream slab can likely be excluded, but potentially unstable blocks under earthquake loading may exist. These virtual blocks are delimited: in the vertical longitudinal direction by the vertical longitudinal cracking and the upstream face (2.50 to 4.10 m), in the vertical transversal direction by dilatation joints and saw cuts (5.30 to 16.30 m) and by cracks potentially linking existing drains (singular points with an average distance of about 3 m) and in the horizontal direction by the construction joints (2.40 to 2.90 m).

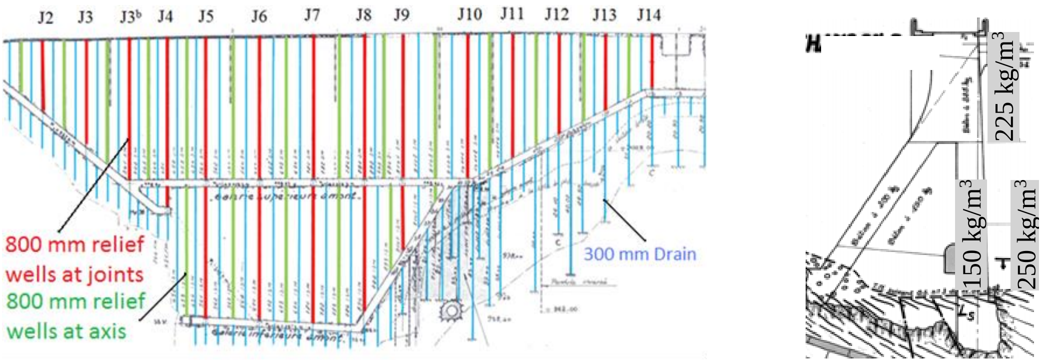

Figure 2. The vertical drainage curtain seen from downstream and cross section

Between structural cracks, the concrete displays good mechanical properties: compressive strength greater than 20 MPa, modulus of instantaneous deformation greater than 20 GPa. Moreover, the contact between limestone (Trias) and igneous rock (gneiss) in the left bank abutment is not disturbed by the thrust of the dam: it is closed and of good quality (with deformation moduli varying between 6 and 14 GPa) excluding any risk of shearing surface development inside the left bank abutment under the stresses developed by the dam swelling.

The swelling laboratory tests showed that the expansion of concrete should continue with a relatively constant rate for several decades, even if a gradual slowing is not excluded (Figure 3).

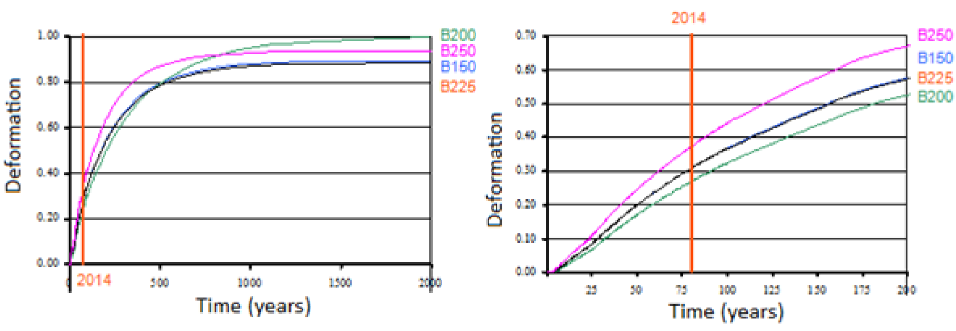

Figure 3. The predicted swelling in the future at Chambon dam

A new multi-scales swelling law for concrete implemented in ASTER computer code developed at EDF was used at this occasion (reference: Grimal E. & al. (2017), “AAR and DEF structural effects modeling”, Swelling Concrete in Dams and Hydraulic Structures, iSTE/Wiley, pp. 203–217). It provided a very good fitting with the monitored dam behavior (Figure 4). The FEM calculations showed that the benefits of the slot cuts done in the 1990's still remained in the upper part of the structure, confirmed by the monitoring of the deformations in the curved left wing.

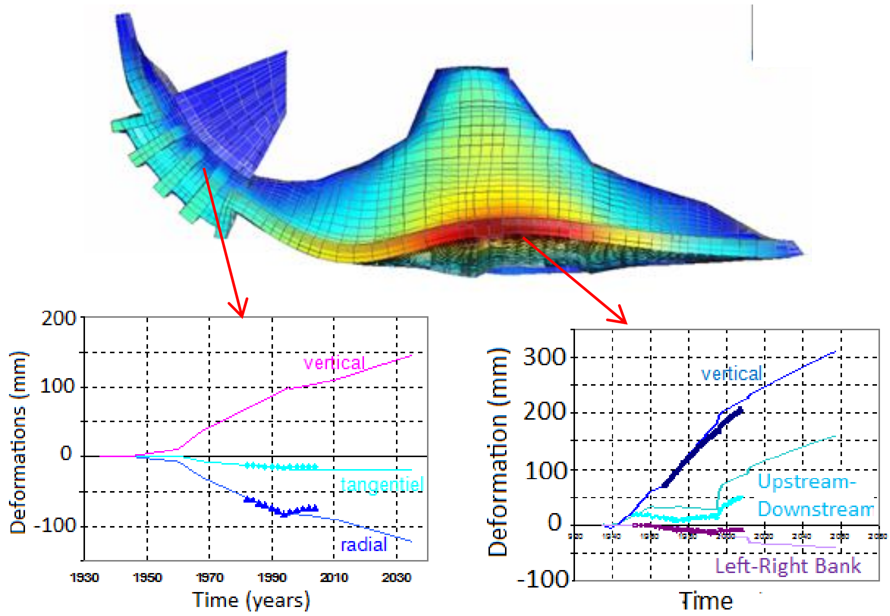

Figure 4. Comparison of measured and computed deformations in 3D modeling

They nevertheless displayed noticeable stresses parallel to the abutments, with the risk of shearing the rock/dam interface in a medium term (Figure 5).

Based on that diagnosis, a new reinforcement campaign was decided and received in 2010 the agreement from the Permanent Technical Committee on Dams and Hydraulic Structures (commissioned by the French Ministry of Industry).

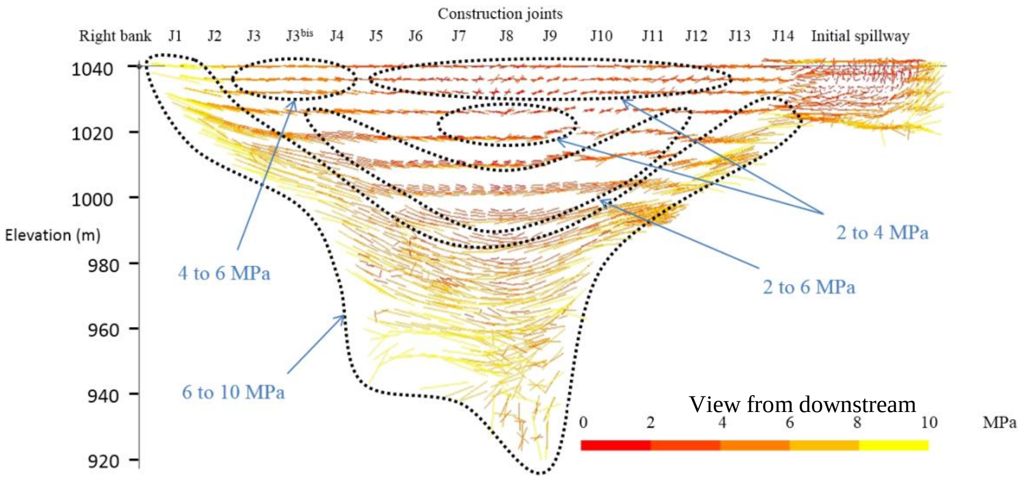

Figure 5. The maximal principal compressive stresses calculated in the dam

4. THE SECOND REINFORCEMENT WORKS CAMPAIGN (20013-2014)

The main objectives of the second campaign were to reinforce the integrity of the upper part of the dam and to prevent any upstream block from falling that could lead to destabilization of the dam. The upper part confinement was obtained by the installation of 415 tendons (Figure 6).

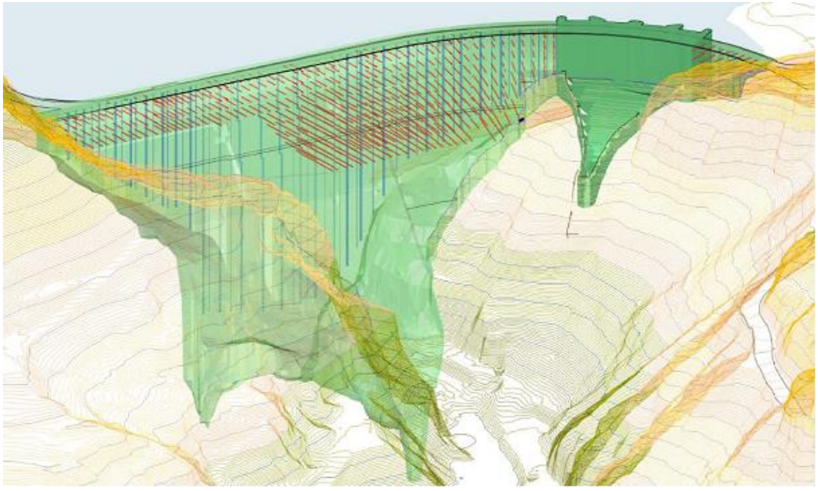

Figure 6. The tendons pattern (red) and drainage curtain (blue) seen from downstream

Consisting of horizontal cables, with greased sheathed strands type T15, crossing the structure from upstream to downstream, they were pre-tensioned and non-grouted on the outer side of the general sheath, as the successful completion of the work could be impaired due to the presence of cracks. They are 3.70 m (horizontal) and 4 m (vertical) spaced, with about one tendon for 15 m². The obtained mesh fits the most likely size of the blocks, delimited by vertical cracks parallel to the axis and the upstream face, vertical joints or former slot cuts parallel to the stream direction and the lift joints in the horizontal direction. Upstream and downstream cable heads are fully embedded in reservations drilled in the dam, with diameter ranging between 500 and 700 mm. The corrosion protection is enhanced on the upstream side by concreting the reservations and by the presence of the sealing membrane. As progressive tension increase due to concrete expansion remains possible, the downstream heads were designed to be adjustable and detensioning operations could be necessary in order to fit the required tension.

The tension of 66 tendons is monitored during dam operation. They are instrumented with sensors, each containing three vibrating wires, connected to the existing remote monitoring system of the dam. Punctual in-situ weighing of each tendon is also possible at any time. Downstream cable heads were designed to maintain their tension within specified adequate limits in any circumstances:

- a lower limit equal to the pulling force due to earthquake, thus preventing any block movement,
- an upper limit of 80% of the tendon yield stress (including earthquake forces, additional tension due to concrete swelling, differential thermal expansion between steel and concrete, as well as measurement uncertainties).

The reinforcement project was designed for a 50 years duration, with a theoretical tendons detensionning every 20 years (representing a theoretical elongation of 24 mm for the longest tendons). Tension design includes the following stresses:

- the pull forces due to earthquake: horizontal acceleration (0.18 g peak ground acceleration) combined with a maximal amplification at crest of about 7 and the thickness of the blocks to confine,
- further swelling concrete, considering isotropic rate of 50 µm/m/year.

The implemented tendons type varies from 3T15 in the lower part (length: 24 m, 369 kN $\leq$ T $\leq$ 595 kN), to 7T15 in the upper part (length: 5 m, 1,060 $\leq$ T $\leq$ 1,388 kN), locally 10T15. The 4180 m drill holes necessary for the tendons were cored from upstream, with a tolerance of 1% deflection. They were systematically inspected by logging digital in order to map the cracks network and to represent them in a 3D digital model.

In addition to tendons, a carbon fiber composite net has been set up on the upstream face. It consisted in the sticking of 6,000 m carbon fiberstrips. The 20 to 30 cm wide strips link the tendon heads, along vertical, horizontal and diagonal lines (Figure 7).

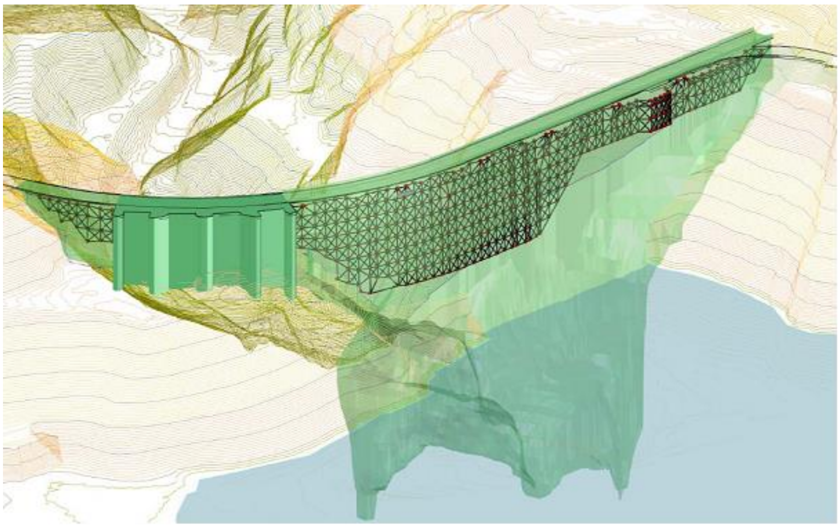

Figure 7. The carbon fiber net seen from upstream

The carbon fiber net serves confinement of small blocks that could escape the tendons action. Strips composite material (carbon strips glued with epoxy resin), composed of 1 to 8 layers glued to the surface previously sand blasted, were designed to form a “chainstitch” and to resist tensile stress due to earthquake and further swelling. An anchoring device, designed after laboratory tests and inserted into the upstream reservation, allows the connection between the carbon fiber strips and the tendon upstream heads (Figure 8). It is composed of an upstream plate maintaining the bands and a downstream plate transmitting the efforts to the tendon head, linked by eight rods (type M24 to M40 according to the efforts to be transmitted). Bands are continuous in current zone and re-curved over the anchoring devices located on the periphery of the reinforced zone.

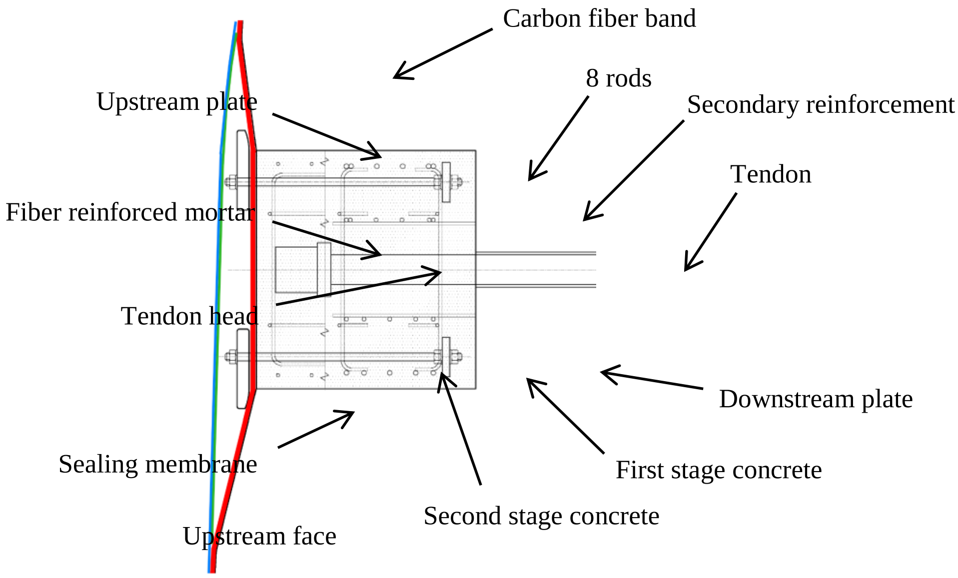

Figure 8. Carbon fiber net anchoring device

While carbon fiber composite is a material commonly used in civil engineering, its implementation as an anti-seismic net is relatively innovative and requested qualification tests in laboratory. The test device was composed of ten concrete blocks simulating tendon head hooped recesses at scale 1 and defining the average mesh (3.70 m x 4 m), connected two-by-two by the test composite bands in different configurations encountered. The tests consisted in applying a force perpendicular to the carbon fiber band. The resistance values from the trials were then included in the whole design and taken into account in a calculation model to determine the number of layers required for each band.

The new campaign by diamond wire sawing was defined using the numerical model (i) to avoid the re-compression of the upper part of the structure and (ii) to decompress the stress paths parallel to the abutments (Figure 9).

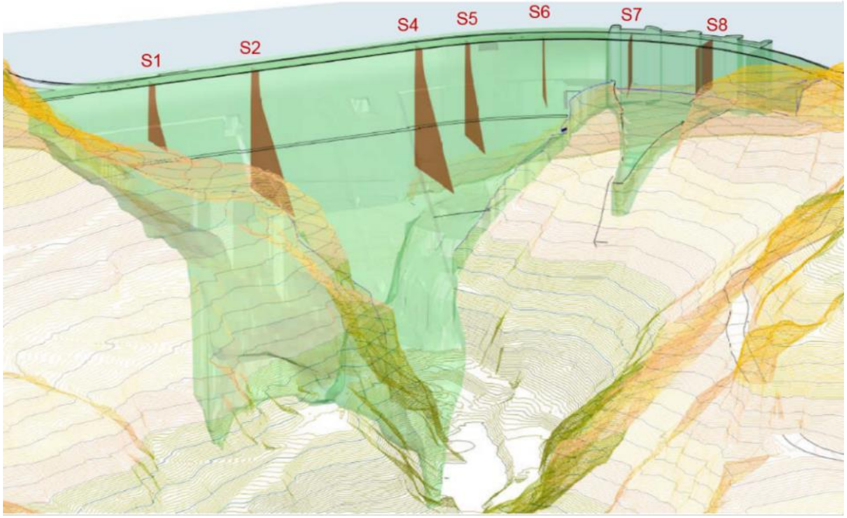

Figure 9. The design of the new campaign of slot cutting

Slot cuts mostly followed the same tracks than in the 1990’s, with the following exceptions:

- slot cut S3 in the central part of the dam was not sawn again, due the presence of a cable stuck during the previous campaign,

- both slot cuts flanking S3 (S2 and S4) were deepened to 42 m, leading to an area of 650 m² each, in order to compensate the non-resawing of S3, but mainly to increase the stress reduction efficiency along the concrete/rock interface,

- the diamond wire was 16 mm wide (against 11 previously) to take advantage of technological progress made since the 1990’s and avoid any risk of jamming in the event of non-fully closed former slot cuts,

- progresses in slot cutting technology allow nowadays far greater rates of cutting and made possible the 2,500 m² total cut in six months, with two equipments.

About 4 years after the works, it is now possible to estimate the influence of the slot cuts on the behavior of the dam. In the upper part of the dam, the effects were very similar to those observed after the first campaign: repositioning of the curved zone toward upstream, acceleration of the central zone and right bank side deformation toward downstream, reopening of the former construction joints and the slot cuts, return of the side blocks toward the center of the dam. In the lower part of the dam, the repositioning of the side blocks toward the center of the dam was significantly greater following the new campaign, confirming the effectiveness of S2 and S4 deepening on the stresses along the abutments.

Regarding the conditions for carrying out the works, several main on-site contrainst have conditioned the procedures and the project schedule (presence of an important roadway on the dam’s crest, altitude: over 1,000 m, situation in an environmentally sensitive area, ongoing operation of the hydraulic power plant, tight schedule with many interfaces between different types of works).

Access to the work sites were performed thanks to scaffolding of the entire surface concerned by the works on both upstream and downstream faces of the dam, representing a decking total length of more than 8,000 m. Two 75 m jib tower cranes were the main lifting equipments for moving up to 3 tons loads over the entire site.

The works lasted from January 2013 to December 2014. The schedule fit the reservoir low water supply, situated in winter, as the river’s regime depends mainly on the snowmelt in spring. The first winter was devoted to the removal of 8,700 m² of the existing upstream sealing membrane. The second winter was dedicated to treating lower areas, by taking advantage of the natural seasonal lowering of the reservoir, limiting the energy losses.
During the works and the requalification of the dam, the reservoir was lowered by 30 m. A preventive volume for storing a 10 years flood was also maintained under the lowest point of the works.

The new waterproofing membrane is conceptually identical to the one completed in 1995, with a few small modifications and improvements. It is tensioned by vertical fixings spaced 1.85 m apart, drained and divided into 12 independent compartments. The behavior of the previous geomembrane remained satisfactory after nearly twenty years of operation and gave the opportunity to re-use a large part of the stainless steel components.

Works were implemented by Bouygues-VSL company, France (tendons, carbon fiber, civil engineering and facilities), Marietta spa, Italy (slot cutting) and Carpi Tech BV, Switzerland (sealing membrane).

5. CONCLUSIONS

The slot cutting works, the installation of tendons and carbon bands, the removal and re-installation of the waterproofing system lasted two years, from January 2013 to December 2014. The good planning, strict coordination of the different tasks, that made these works so unusual, the good cooperation between EDF and the various contractors involved, allowed the works completion successfully and within the deadline.

Reservoir impounding was successfully operated during 2016 spring period, with one year of delay due to the reactivation of a landslide overlooking the reservoir. This event had nothing to do with the remedial works. The extended monitoring system enables to follow precisely the dam behavior and confirms that the effects of the slot cuttings are in line with the model predictions. Regular diagnostics will continue to be carried out in order to ensure that the dam operation continues under optimal safety conditions.