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Feedback from the bituminous geomembrane (BGM) implemented 20 years ago at the Galaube dam

N. Pepin & L. Deroo
ISL Ingénierie, France

B. Breul
Axter, France

B. Gasc
Institution des Eaux de la Montagne Noire, France

ABSTRACT: The Galaube dam is a 40 m high French rockfill dam commissioned in 2001. The upstream facing is ensured by a 4.6 mm thick bituminous geomembrane (BGM) directly laid on a cold mix layer and an impregnated layer gavel.

During construction, control measurements were first carried out in the factory. The BGM was then installed by vertical strips, in one piece, from top to bottom. Welds were subject to a strict quality control plan and all defect immediately rectified.

After 20 years of operation, the hydraulic behavior of the dam shows that piezometry is low and leakage through the membrane is deemed to be negligible. The dam has settled very few in practice and has already undergone its first filling and a complete emptying in 2007. No defects have been identified and almost no maintenance operations carried out. The design and construction of the upstream facing has been very successful.

1 INTRODUCTION

The Galaube dam on the Alzeau river, located in the communes of Lacombe (Aude department, France) and Arfons (Tarn department, France), was built in 1999-2000. It is operated by the Institution des Eaux de la Montagne Noire and was designed to store and regulate water resources for drinking water, irrigation and navigation on the Canal du Midi.

The dam is a rockfill dam with a tight upstream facing. The maximum height of the structure on the natural ground is 33 m. It creates a reservoir with a volume of 7.7 hm^3 below the normal reservoir level of 720.5 m NGF. The structure is class A according to French legislation on the safety of hydraulic structures.

The purpose of this article is to analyze the behavior of the dam after more than 20 years of operation, and more particularly of the bituminous geomembrane (BGM) which ensures the upstream sealing of the structure. We summarize the reasons for the choice of a rockfill dam, then describe the characteristics of the structure and its waterproofing membrane. Finally, we analyze the hydraulic and mechanical behavior of the dam. Our study indicates that the BGM waterproofing system is very effective.

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2 CHARACTERSITICS OF THE DAM AND DESIGN OF THE UPSTREAM FACING

2.1 *Initial design options at the Galaube dam*

Two possibilities were studied and compared at the Basic Preliminary Design stage for the Galaube dam: a roller-compacted concrete dam (RCC) or a rockfill dam with an upstream face. The rockfill dam was finally selected by the project designers because it made the most of the poor foundation and could be easily adapted to the lack of watertight materials by adding a specific watertight component.

Two solutions were possible, depending on the position of the watertight component: an upstream facing or a central core dam, either with an asphalt core or a plastic concrete core. The rockfill dam solution with an upstream facing was considered the most suitable for the Galaube dam because of the following advantages: it was less expensive (in this case), there were lower risks of internal erosion of the sandy gouges of the foundation thanks to a better connection with the granite foundation, there was better handling of the interface with hydraulic galleries and, construction schedule was more flexible.

2.2 *Design of the upstream facing*

2.2.1 *Cross-section of the dam*

The typical cross-section of the dam is shown in Figure 1. The rockfill consists of two main zones:

- an upstream zone with a horizontal width of 3 m (Zone 2), consisting of a material with a grain size of 0/150 mm, ensuring a triple function: *i)* a transition zone between the rockfill and the support layers of the geomembrane, *ii)* a semi-permeable zone limiting leaks in the event of a localized defect on the upstream facing, and *iii)* the creation of a contrast in permeability with the downstream rockfill. The latter contributes to the lowering of the water line in the event of accidental loading by damage to the upstream facing or accidental impounding during construction.
- the compacted rock fill 0/500 mm (zone 3).
- the reinforced concrete plinth is founded and anchored on fresh or slightly weathered granite. A grout curtain ensures that percolations in the foundation are cut off.

Prior to the placement of any material of the upstream facing, the upstream slope of the dam embankment was compacted with two static rollers of 4 tons each, specially designed and manufactured, and pulled from the dam crest. This compaction complemented the previous compaction of the backfill.

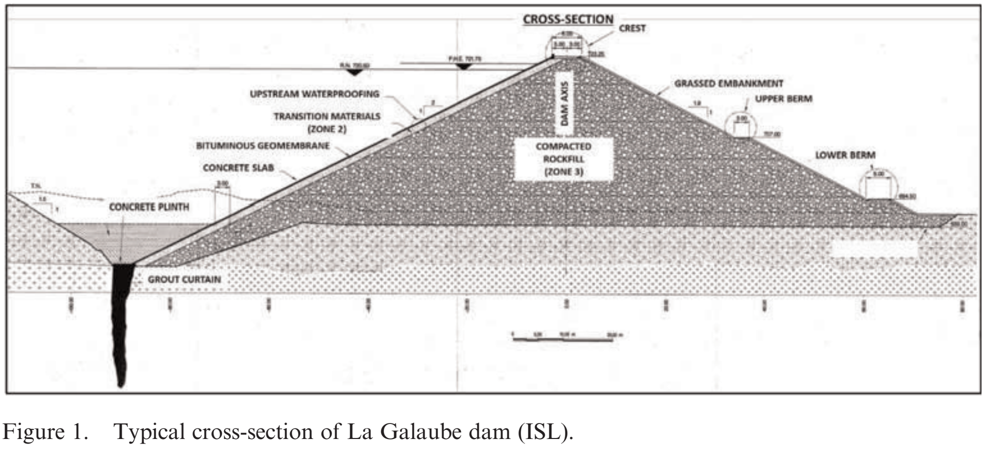

Figure 1. Typical cross-section of La Galaube dam (ISL).

2.2.2 *Technical specifications of the upstream facing*

The typical cross-section of the upstream facing of the Galaube dam is shown in Figure 2.

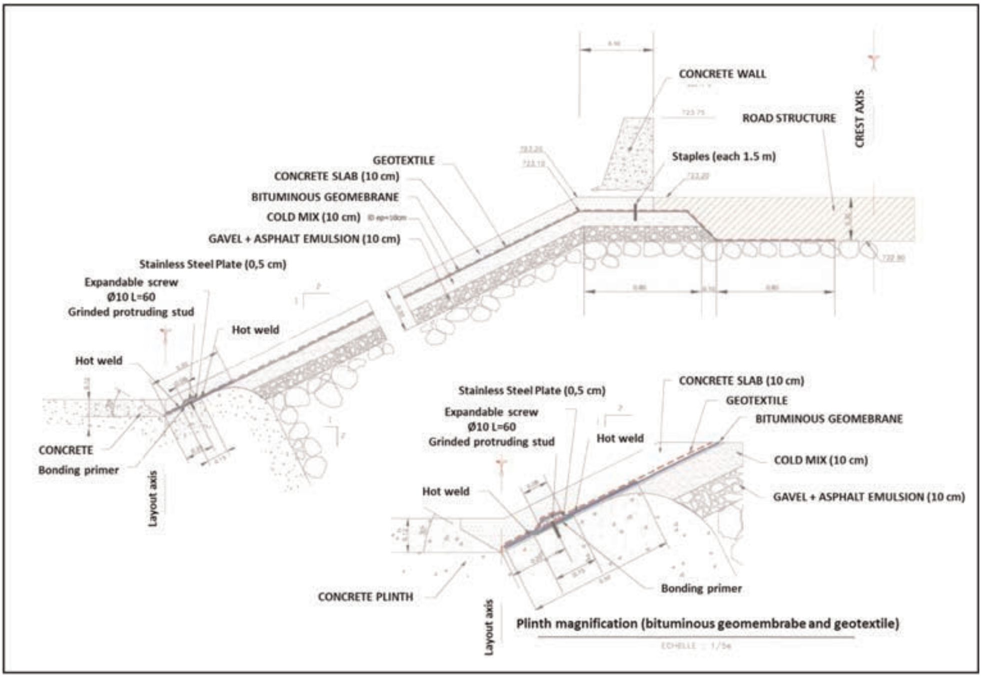

Figure 2. Typical cross-section of the upstream facing.

2.2.2.1 *Membrane supporting layers*

The membrane supporting layer consists of two sub-layers:

– a layer of gravel, with a grain size of 0/20 mm, with a minimum thickness of 10 cm impregnated with bitumen. It is designed to absorb most of the irregularities in the flatness of the dam body and those that may occur because of prior compaction of the slope.
– a cold semi-permeable asphalt mix layer with a nominal thickness of 10 cm and permeability between $0{,}75 \times 10^{-6}$ et $1{,}25\ 10^{-6}$ m/s. The controlled breaking behaviour of the emulsion was selected so that the cold asphalt mix manoeuvrability was ensured throughout the implementation and compaction phases.

2.2.2.2 *Bituminous geomembrane*

The upstream facing of the dam is ensured by a 4.6 mm thick “Coletanche NTP3” bituminous geomembrane with the following data:

– Mass per unit area of 5.5 kg/m2
– High tensile strength in both directions: 28 kN/m in longitudinal direction, 20 in transverse direction, together with more than 70 % elongation at break, according to French standard NF P 84-50
– High resistance to puncture: 500 N according to French standard NF P 84-507
– tensile/shear joint strength greater than 75% of the tensile strength in the current part, according to French standard NF P 84-502.1.

A series of measurements on each of the components was carried out in the factory for each laid strip. The membrane was laid directly on the cold mix layer without the interposition of a geotextile and by following a detailed installation drawing. The membrane was unrolled from top to bottom and each strip was laid in one piece from the crest attachment to the weld on the concrete plinth. Horizontal joints were not permitted.

Welds were performed according to the procedure recommended by the geomembrane manufacturer. The minimum overlap to be achieved by welding was 20 cm. Welds should only be carried out on perfectly clean surfaces, free of dust, moisture, and grease. A continuous control of all the welds between the strips was carried out. For the Galaube dam, the weld quality was checked by a continuous non-destructive device called CAC 94. This machine had a measure wheel, which included 24 ultrasonic sensors that could detect flaws of a minimum surface of 0.8×0.5 cm at the interface between the two geomembranes, on a 21 cm wide strip. If a section of welds showed defects, repairs had to be carried out immediately. A wider patch with at least 30 cm around the flaw was then applied, hot welded and checked again.

The concrete plinth was anchored into the rock foundation. The geomembrane was fastened to the plinth at the foot of the slope and along the whole periphery of the impervious face. It was hot-welded on the 2H/1V sloping part of the concrete surface, which has been previously covered with a tack-primer, then anchored with stainless steel plates bolted into the plinth, each 15 cm.

At the crest, the geomembrane was clamped using metal staples spaced 1.5 m apart, driven into the cold mix asphalt layer and into the concrete slabs overlying the membrane. The membrane was flipped under the road structure over a length of 0.80 m.

For the cconnection to concrete structures, the materials used were the same as those used for fixing to the concrete plinth.

2.2.2.3 Geotextile

A geotextile was placed above the BGM: it provided a puncture protection function when the protective slabs were laid and now ensures a drainage function for the underside of these slabs. The geotextile presented a mass per unit area of 500 g/m^2 (NF G 38-013), a tensile strength of 20 kN/m (NF G 38-014), and a transmissivity higher than 10^{-6} m^2/s (NF G 38-018).

2.2.2.4 Concrete slabs

The dimensions of the protective slabs for the geomembrane were 5 m width (horizontal) and 10 m length (along the slope). The concrete (35 MPa standard compressive strength) was cast in place. The anti-cracking device was made of polypropylene fibres (0.4% by volume) which tensile strength had to be greater than or equal to 500 MPa.

Horizontal joints were continuous and consisted of simple concrete interfaces. Vertical joints were staggered and designed as dilatation joints and filled with a strip of expanded polystyrene type material of about 20 mm. After concreting and hardening of the second panel, they were cleared of the joint material and left open.

The horizontal and vertical joints between minimum and maximum water level were filled with a flexible mastic to prevent the joints from being filled with silt and vegetation.

3 FEEDBACK ON THE MONITORING OF THE UPSTREAM FACING

3.1 *Analysis of the hydraulic behavior of the dam*

The filling of the reservoir took place in two seasons. In June 2001, the reservoir reached a maximum level of 710.94; the level then progressively decreased until it reached 704.50 in December 2001. In February 2002, the reservoir exceeded the maximum level reached in 2001. The normal reservoir level of 720.50 was finally reached in June 2002. The reservoir had an average annual tidal range of 15 m until the 2007 drawdown. Afterwards, the range has been about 5 to 10 m until today.

The piezometry at the downstream foot of the plinth (Figures 3 and 6) appears to be well draw down with a low (less than 30%) to very low (less than 10%) dependence on the reservoir level. Only PF03 piezometer shows particularly sensitive to the level of the reservoir

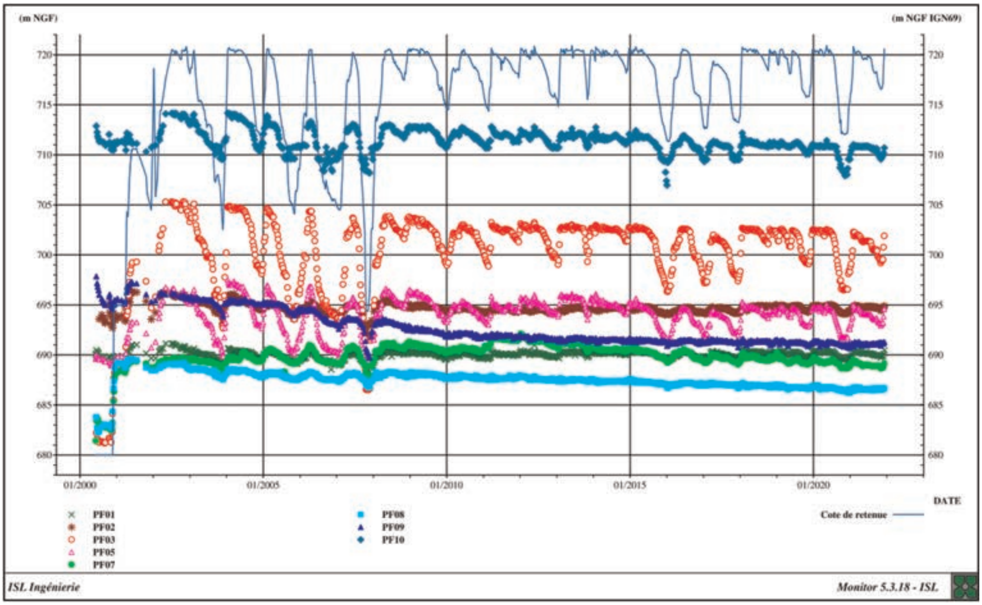

Figure 3. Time evolution of the piezometric levels in the foundation, downstream of the plinth.

and reacts by faithfully reproducing the reservoir curve. This singular behavior, detected immediately after the first filling, is not reproduced by its neighbors (PF07 and PF08). As PF03 is drilled in a zone where the rock is of good quality and low permeability, it is likely that PF03 measures a singularity of piezometry, for example by capturing a fine fissure, where pressures can become high due to the low permeability of the medium. Watertightness of the geomembrane is therefore not in question.

Overall, the analysis of piezometric levels downstream of the plinth over the period 2001–2021, shows that they are constantly decreasing since impoundment, on average by about −10 cm/year. It is most certainly a consequence of the progressive sealing of the upstream foot of the dam in connection with the progressive silting of the reservoir.

In the axis of the dam (Figure 4), PF06 and PR03 were apparently above the piezometric level at the first filling but they have shown a continuous decrease since then confirming the very good behaviour of the upstream facing. The piezometry at the bottom of the valley (Figure 4, PR01, PR02, PF04), is quite homogeneous under 690 m NGF, which corresponds to the foundation level of the dam in this area. The rockfill, which is very draining, does not allow piezometry to develop.

Regarding the piezometry downstream of the dam (Figure 5), only the piezometers at the top of the bank (PD01, PG01 and PG02) reacted to first filling and show now a strong dependence on the reservoir level. These piezometers are the closest to the reservoir and they are located in an area (the upper bank) where the foundation is permeable (upper left bank made up of deeply altered granite, and upper right bank where there is an interface between the granite foundation of the dam and the shale that overhangs it). All these piezometers are traditionally influenced by weather conditions, especially the heaviest rainfall events which lead to reversible peaks.

With respects to the leakage flows, the absolute value of the measured flows at the end of impoundment was of the same order of magnitude as what had been observed on other dams of the same type in France. Given the greater height of the Galaube dam, and especially the greater dammed surface area than that of these previous dams, the level of measured runoff was considered satisfactory (0.7 liters per hour and per square meter of facing). Leakage

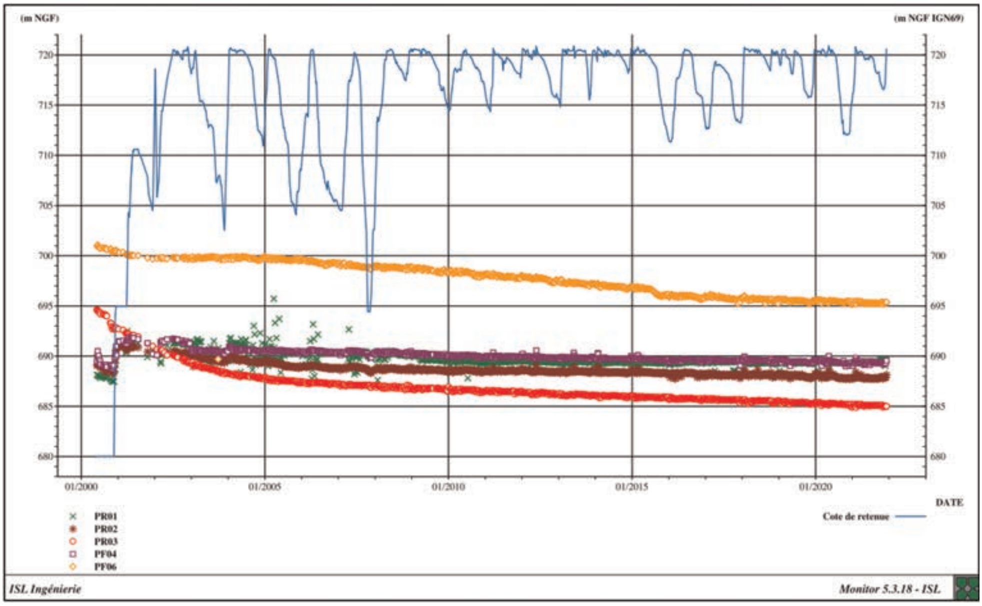

Figure 4. Time evolution of the piezometric levels at the interface of the dam body and the foundation.

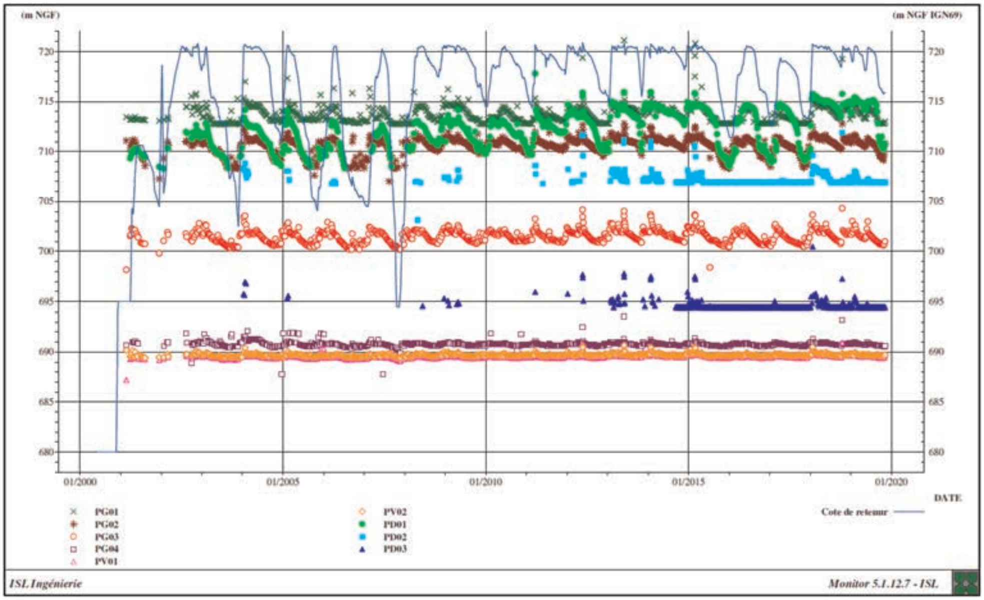

Figure 5. Time evolution of piezometric levels downstream of the dam and on the riverbank.

rates have remained generally moderate – on average about 20 l/min for the flow drained by the drainage gallery – which is well correlated to the reservoir level and increases significantly above 720 m NGF. The flow drained by the temporary diversion tunnel on the left bank – on average about 60 l/min – is also dependent on the reservoir level, but it has a higher dispersion, mainly due to rainfall.

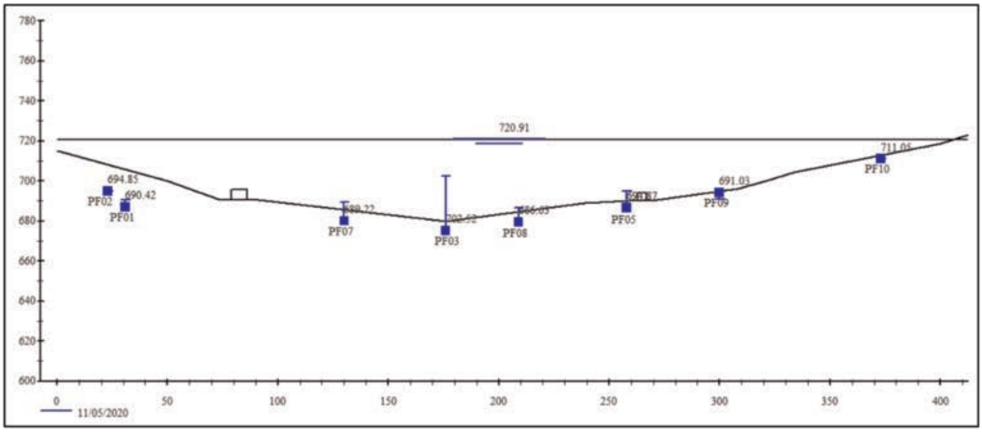

Figure 6. Maximum piezometry in the foundation - downstream of the plinth (period 2018-2019).

The monitoring of leakage flows over the period 2001–2021 shows a continued favourable downward trend of almost −2 l/min/year. This confirms the good watertightness of the upstream facing and the absence of leaks.

3.2 *Analysis of the mechanical behaviour of the concrete plinth and upstream facing*

3.2.1 *Deformations of the upstream facing*

Calculations of the deformation of the dam's upstream facing under the effect of the structure's own weight and the water pressure exerted by the reservoir were carried out in 1996 when the dam was designed.

The calculated deformations of the facing during the operation of the reservoir show that the most stressed area of the membrane is located near the plinth where the differential settlements created by the variation in the height of the embankment cause the geomembrane to be put under tension.

Careful excavation and plinth geometry aimed to limit discontinuities in the upstream facing foundation to limit localized differential settlement. The same applies to the connection of the facing to the upstream structure and to the spillway, where 0.5H/1V facing slopes have been provided to limit membrane extension deformations. These slopes also made it possible to ensure satisfactory compaction in the entering angles near the concrete facings.

The expected membrane deformations – of the order of a few millimetres per metre at most – are negligible compared to the specified elongations at failure of the membrane, which are greater than 40%. The weight of sediment in the lower part of the upstream facing does not seem to be able to alter these conclusions. The deformations generated by the incremental vertical stress (of the same order of magnitude as those calculated under the effect of hydrostatic pressure alone) are in fact considered negligible compared to those generated by possible differential settlement of the fill, which are themselves considered very unlikely in view of the deformations:

- The proven behavior of the structure: it has settled very few in practice (cumulative settlement less than 4 cm at the crest, i.e. less than 0.1% of the total height of the dam) and less than most upstream facing rockfill structures of the same type (see comparison in the graphs on Figure 7 and Figure 8 with 11 well compacted dams, i.e. with materials of medium to high quality $20 < RC < 70$ MPa) ;
- The precautions taken during the construction phase to compact the rockfill, in particular: a maximum layer thickness of 80 cm, a minimum of 8 passes (one way) of a heavy smooth

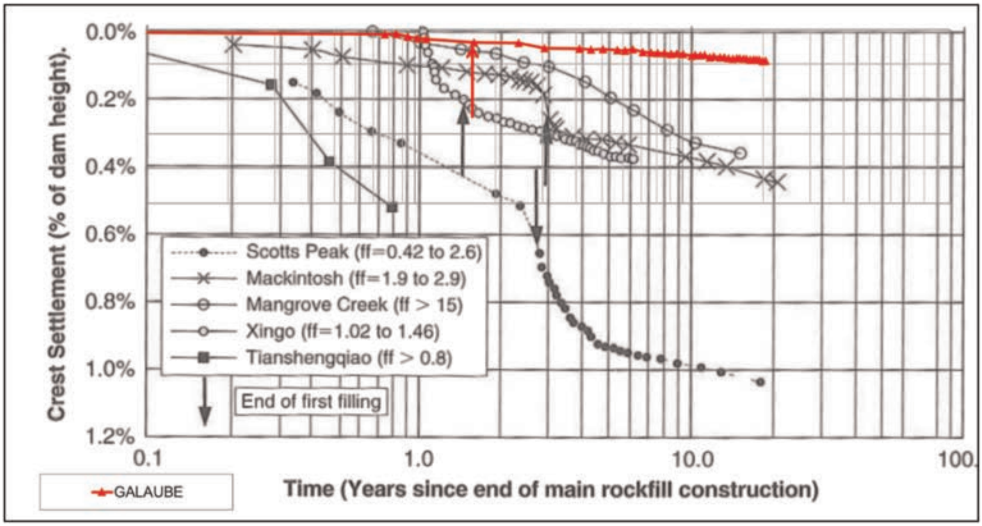

Figure 7. Comparison of maximum crest settlements in relation to 5 dams of the same type, some of which were sensitive to first filling.

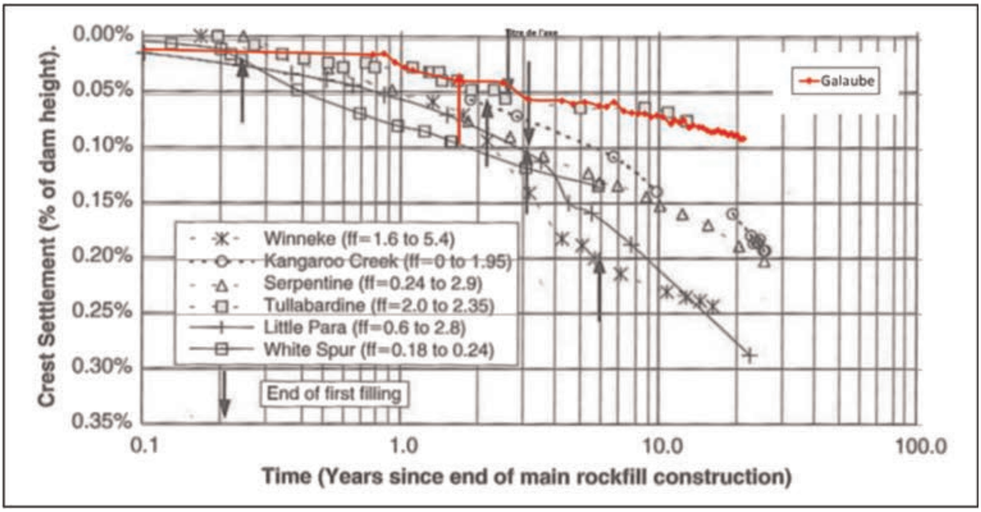

Figure 8. Comparison of maximum crest settlements in relation to 6 other dams of the same type.

vibratory roll, a travel speed of less than 3 km/h, specific compaction in the vicinity of concrete structures (using mechanical tampers), specific compaction in the vicinity of the plinth with a backfill built directly above the structure to ensure optimal compaction, the excess volume being removed before the membrane support layers are placed, and a strict control plan, particularly in the transition zones under the upstream facing (geometry, granulometry, density weight and permeability).

3.2.2 *Effect of welding*

The welding of geomembrane panels is an essential aspect for the success of their implementation as an impermeable barrier. For the Galaube dam, the geomembrane was installed by vertical strips in a single piece, so there are no horizontal joints. The vertical joints were subjected to a strict quality control plan. All welding defects were immediately rectified by welding a piece of geomembrane that was larger than the identified defect. One should thus expect criteria at break close to those of the geomembrane itself (elongation at break $\geq$ 60%).

3.3.3 *Risk of puncturing*

Geomembranes, especially polymeric geomembranes, placed on any type of aggressive rough substrate or soil containing stones, pieces of wood or other debris are vulnerable to puncturing stresses, which may cause leakage. ICOLD Bulletin No. 135 states that "*the lining with a needle-punched non-woven geotextile, whether above or below, improves puncture resistance quite impressively*." The risk of puncture can occur at three different points in the life of the liner:

– During the installation of the geomembrane (tool falls, snags and wear caused by site workers and equipment, etc.),
– During the installation of the covering layer (the most crucial step, regardless of the sealing system),
– After impounding, due to the hydrostatic pressure pushing the geomembrane against protrusions of the support.

From this point of view, the Galaube dam has many favourable factors:

– A geotextile is placed above the membrane: it provides a function of protection against puncturing during the installation of the protection slabs and a function of drainage of the underside of these slabs.
– The bituminous membrane is laid directly on the cold mix layer without the interposition of a geotextile because the layer is smooth (gravel with a regular grading curve of between 0 and 10 mm) and is itself laid on a transition layer consisting of a 20 mm minus well-graded gravel.
– The dam has already undergone its first filling, a complete emptying in 2007, as well as 20 years of service without any observed defects.

3.2.4 *Ageing of the geomembrane*

Geomembranes are sensitive to UV radiation, thermal, biological or chemical aggression. However, ICOLD Bulletin No. 135 states that: "*It goes without saying that the lifetime of protected geomembranes will be much longer than that of exposed geomembranes. It seems reasonable to expect a geomembrane protected by a permanent concrete layer to have a life expectancy of more than 200 years.*"

At the present time, there is no reason to suspect any issue with the BGM of the Galaube dam; leakage through the geomembrane is deemed to be negligible.

3.3 *Lessons from 2007 drawdown*

3.3.1 *Behaviour of the dam during the drawdown*

The reservoir of the Galaube dam was emptied for the first time in the fall of 2007 as part of the first "five-year" inspection of the dam, which generally takes place 5 to 6 years after the dam is impounded according to French regulations.

The inspection reports n°3 (February 2008) and n°4 (February 2009) prepared by ISL Ingénierie) did not reveal any anomalies in the hydraulic and mechanical behaviour of the structure, which proved to be completely reversible.

It should be noted that a slight elevation of the dam crest was recorded (of the order of 1 to 2 mm and visible on all the height benchmarks on the crest of the dam), during the emptying (cf. Figure 9). This is a mechanical consequence of the decrease in hydrostatic

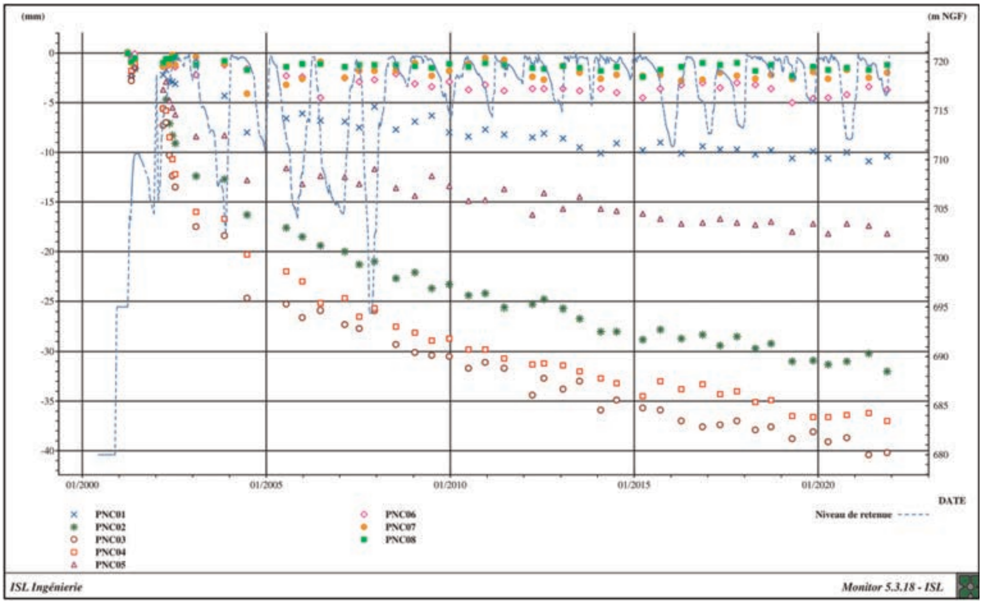

Figure 9. Time evolution of crest elevation measurements.

pressures of the reservoir on the upstream face of the structure (decompression of the rockfill).

It was also checked that the under-pressure behind the facing remained below the hydrostatic pressure exerted by the reservoir on the facing, which did not create a risk of lifting the geomembrane or the concrete protection slabs (cf. Figure 10).

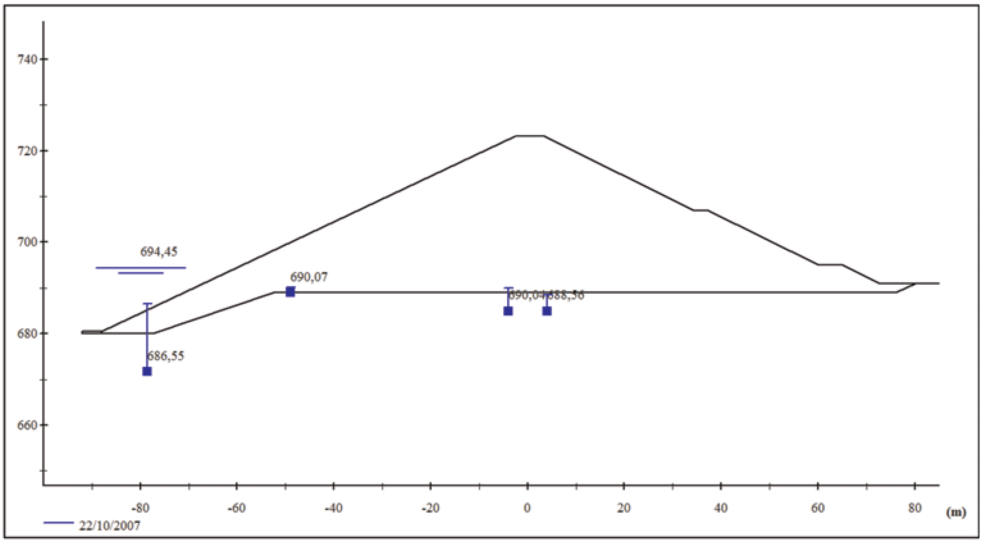

Figure 10. Maximum piezometry downstream of the plinth at the time of the 2007 reservoir emptying.

3.3.2 *Maintenance work carried out on the upstream facing*

The mechanical protection of the facing membrane of the Galaube dam consists of juxtaposed 5 m wide × 10 m long slabs cast in place with a fibre-reinforced concrete. The horizontal joints are parallel and the vertical joints are offset by half a slab. An expansion joint made with expanded polystyrene was present on the periphery of each slab, after the drying and shrinking of the concrete. It was observed that this joint was no longer held and it was torn off by wave action. Silt had begun to settle in this void and vegetation to grow.

A first area in the upper part of the upstream face had been treated during the construction site (a row and a half of slabs vertically and from bank to bank horizontally), but despite this, as the water level was lowered, plant growth began to take root in the unfilled spaces.

The owner and operator of the Galaube dam (IEMN) decided to extend the filling of the joints to the lower slabs with identical product, so that the usual tidal height is now completely treated. The work was carried out as part of the 2007 drawdown.

The horizontal joints have been filled in. For the vertical joints, a gap of approximately 20 cm was left at the bottom of each joint to allow water that had passed behind the plate to drain away as the level of the reservoir was lowered. The product used was cold bituminous mastic applicable even under water: Shell Tixophalte Wet.

This is the only work carried out on the upstream face of the Galaube dam during the first 20 years of the structure's life. Underwater inspections conducted in October 2020 as part of the exhaustive review of the dam thus showed no adverse changes to the upstream facing.

4 CONCLUSION

All the elements presented in this article allow us to affirm that the BGM upstream facing has been very successful at the Galaube dam: great care in the construction of the structure, low piezometry, absence of leaks and no notable disorders for 20 years of operation. It also has a very good record on other French dams, some of them a bit older than the Galaube dam (though not as high).

Of course, every project still requires adaptations and, despite its great success, the Galaube dam design shall probably not be directly applied to other project. The need from adaptations might come from three reasons:

- The design of an upstream facing depends a lot on the nature of the backfill materials and of the foundation.
- At Galaube dam, the rockfill and the foundation were of poor quality, which led to a gentle 2:1 upstream slope; steeper slopes might be considered – but stability of the upstream facing must be checked carefully.
- The Galaube dam design may be considered as "very safe" and could probably be optimized.