Implemented solutions to enhance the safety of the alluvial foundation of the Crestuma-Lever dam

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Introduction

The Crestuma-Lever dam, located in the North of Portugal, in the fluvio-maritime stretch of the Douro River, is the major element of the most downstream hydropower scheme in the Douro catchment. Completed in 1985, it also provides water supply to the Oporto metropolitan area and plays a key role in river navigation.

The dam is a gate structure type, consisting of 8 spans (7D, 5D, 3D, 1D, 1E, 3E, 5E and 7E) controlled by double fixed wheel gates, 28 m wide and 13.8 m high, and designed to a maximum discharge flood of 26,000 m^3/s. The concrete stilling basins, 28 m wide, 56.6 m long and 6 to 9.5 m high, with the exception of 7D, are resting on the alluvial river bed, which presents a maximum thickness of 40 m. The piers between spans are founded in the bedrock and structurally independent from the basins. In most basins, the upstream and downstream cut-off walls do not reach the bedrock. Upstream and downstream of the stilling basins, the alluvial river bed is protected by a rockfill blanket [Ribeiro et al., 1973, 1976, 1979, 1982].

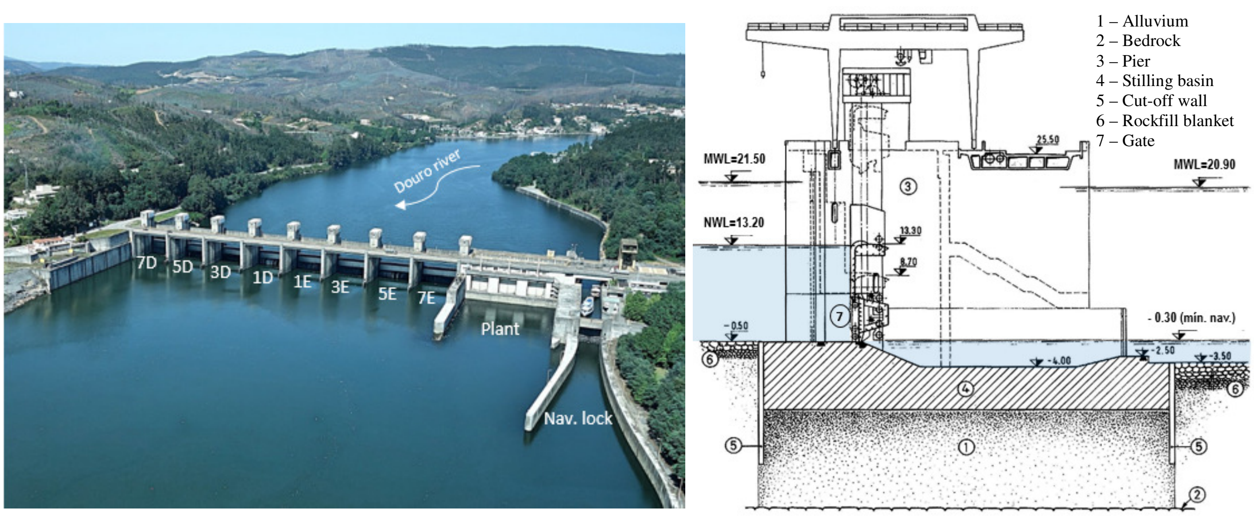


Figure 1 – Aerial view and cross section of the dam.

Corrective measures were implemented in Crestuma-Lever dam, in 2012 and 2013, including the execution of a concrete secant pile curtains, upstream of stilling basins 1E and 3E, and suitable filter and rockfill layers, with rockfill significantly heavier than the original, downstream of the 8 basins [Fernandes et al., 2017].

During those interventions, an upward flow of water was detected immediately downstream of the stilling basin 7E, under a large concrete block, near the wall between the basin and the powerplant. Some measures were immediately implemented to increase the safety conditions of the basin: filling the voids beneath the block with filter material and loading the block with rockfill.

The following phase, involving the diagnosis of the upward flow causes and the safety conditions of stilling basin 7E, included the analysis of documentation concerning construction and operation of the dam, underwater inspections of the basin and a geotechnical investigation.

Based on the results of the geotechnical investigation, a numerical three-dimensional model was generated, for evaluating the seepage conditions under the basin and for predicting their possible evolution in face of an upstream intervention.

The results of the numerical model led to the definition of corrective measures, in order to improve the safety of the basin. These measures were implemented in 2016.

The geotechnical studies, carried out during the diagnosis phase of the upward flow causes, the analysis of seepage through the foundation of basin 7E, as well as the main features of the implemented corrective solutions and quality control of the execution, are presented in the following sections.

1. Geotechnical investigation

The goal of the geotechnical investigation, carried out in 2014, was evaluating the possible existence of voids under the basin and ascertaining the composition, thickness and relative density or consistency of its alluvial foundation, as well as evaluating the pore pressures along its thickness. The goal was also to determine the depth and characteristics of the bedrock and the filling of eventual voids underneath the basin.

For this purpose, 9 boreholes were executed in basin 7E, whose location is represented in Figure 2. To avoid disturbing the pore pressures under the basin and to maintain the equilibrium of those pressures, these boreholes were executed through the interior of steel tubes (with an inner diameter of 165 mm), properly fixed at the slab of the basin and whose top level was above the normal water level in the reservoir.

The upstream boreholes, designated by SM, were executed from a floating platform. The central ones, designated by SC, were executed from the dam's bridge, and the downstream ones, designated by SJ, were executed from a fixed platform installed for that purpose (Figure 2).

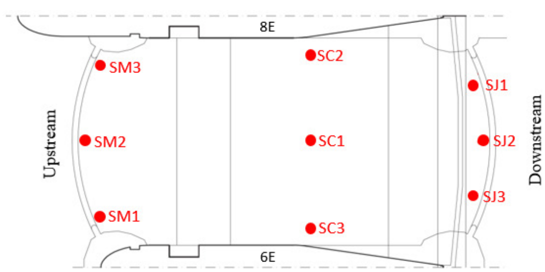


Figure 2 – Boreholes location plan and view of the execution of the downstream boreholes.

The boreholes went first through the reinforced concrete basin, which presents variable thickness (9.5 m upstream, 6 m in the central area and 6.5 m downstream). Afterwards, with a casing tube, the alluvial deposits were crossed and, finally, the bedrock was drilled, penetrating, at least, approximately 3 m of good quality schist.

Envisioning the characterization of the relative density or consistency state, standard penetration tests (SPT) were carried out, during the drilling, in the alluvial layers, also collecting representative samples of the drilled soils. After stabilizing, the pore pressures along the alluvial formation were measured using a plug valve and a pore pressure cell, introduced at the base of the casing tube.

The boreholes revealed that the alluvia formation under the basin has a maximum thickness of 14 m, consisting of mostly superficial muddy soils and silty sandy soils at greater depths.

The muddy soils, for the downstream boreholes SJ1, SJ2 and SJ3, as well as for the central boreholes SC1 and SC3, were characterized, particularly, in the most superficial 3 m, with null or very low values of resistance to dynamic penetration.

Borehole SM1 had no significant presence of muddy soils and, for boreholes SM2 and SM3, the muddy soils presented a greater consistency. Lastly, borehole SC2 intersected, at the surface, mostly sandy soils, for which the corresponding values of penetration resistance were higher as well.

The contact between the alluvial formation and the bedrock occurs at variable elevations between -15.10 m and -20.00 m upstream, between -21.60 m and -23.45 m in the central area and between -18.20 m and -24.05 m downstream.

As the base of the upstream cut-off wall is at elevation -17.50 m, the possibility of seepage through the upstream sandy formation could occur in a zone localized mainly between borehole SM2, at the axis of the basin, and pier 6E. Nevertheless, concrete was detected in borehole SM1, between elevations -14.60 m and -15.85 m, which may be associated with the repair of a panel of the cut-off wall, after its construction, to narrow a detected zone of circulation of water close to pier 6E.

The downstream cut-off wall does not reach the bedrock in all downstream boreholes, which means seepage may be substantial under the cut-off wall.

After the execution of the boreholes, given the presence of voids and the very soft consistency of the superficial soils, concrete and sand fillings were executed.

Regarding the pore pressures measured along the alluvial formation, it was found that, in general, they are controlled by the downstream level of the river and that there is a significant head loss upstream, except for borehole SM1, near pier 6E as explained above, and for borehole SC2.

The underwater inspection of the basin enabled to identify water flow in some areas of the perimeter joint between basin 7E and the upstream cut-off wall. These results confirmed those obtained in the water recovery tests conducted in boreholes SM1 and SM3, which showed the direct communication between the reservoir and the ground under the basin, through the perimeter joint and the existing cracks in the basin.

2. Seepage through the alluvia foundation of the basin 7E

A numerical three-dimensional model was built for evaluating the seepage conditions under basin 7E, as well as predicting their possible evolution in face of an upstream intervention. It was based on the results of the geotechnical investigation described in section 1, particularly identification of the alluvial layers and of the bedrock, as well as of the measured pore pressures.

Numerical modelling was executed using the software *Groundwater Modeling System – GMS v.7.1 of Aquaveo*. It is a finite difference software, which uses a prismatic mesh.

The following simulations were executed in steady state flow:

a) Design situation corresponding to average values of the water level in the reservoir (12.5 m) and downstream (1.0 m) and to average piezometric heads during the geotechnical study;
b) Design situation corresponding to the maximum hydraulic head, with the maximum water level of the reservoir with no discharges (13.2 m) and the minimum water level downstream (-0.3 m);
c) Design situation for the maximum uplift pressures under the basin, with the maximum water level of the reservoir (13.2 m) and downstream (2.0 m);
d) Design situation after the upstream water tightness solution.

The numerical model is shown in Figure 3, covering an area of 87 × 34 m^2, with a mesh of 1 x 1 m^2 in plane view and with variable thickness. The model has a total of 29,580 prismatic cells, distributed in ten layers. The different soil and bedrock layers as well as the concrete structures, considered water tight, are represented. The alluvial foundation was basically divided in muddy soils (mud and muddy sands) and silty sands, and the bedrock was divided in very weathered and fractured schist and less fractured schist.

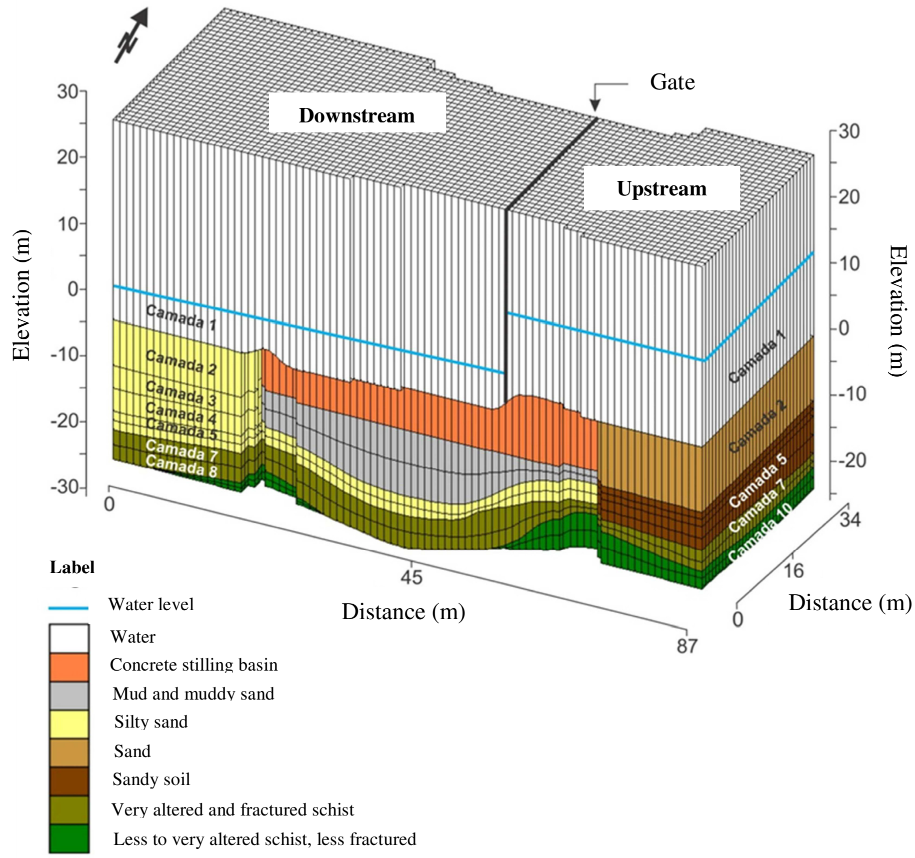


Figure 3 – Finite difference mesh of the numerical model of basin 7E [Caldeira et al., 2015].

As the zone upstream of the basin was in part excavated to execute the power plant wall, it was admitted for the upstream layers a sandier constitution, closer to the materials encountered in the other basins. Therefore, the hydraulic conductivity of the sand was considered 10^{-3} m/s in the more superficial zone and 10^{-4} m/s in the deeper zone, in the latter case an intermediate value between the superficial zone and the one adopted for the silty sand layer. In Table 1, the values admitted for the coefficient of permeability of each material are presented.

Table 1 – Coefficient of permeability, k, of the different layers considered in the model.

Lithology	***k*** (m/s)
Mud and muddy sand	10^{-8}
Silty sand	10^{-5}
Sand	10^{-3}
Sandy soil	10^{-4}
Very altered and fractured schist	10^{-7}
Less to very altered schist, less fractured	10^{-8}

Here, situations b) and d) will be presented. Regarding the former, which corresponds to the maximum hydraulic head, with the maximum water level of the reservoir (13.2 m) and the minimum water level downstream (-0.3 m), the calculations were made in steady state flow. This means that the hydraulic head is not real for the mud formations, because they aren't very sensitive to rapid variations in the water level. Thus, the values presented for the muddy formations are conservative (as well as the ones to the silty sand formations, in a smaller scale) but they constitute upper limits concerning the evaluation of the dam's safety conditions.

The cut-off wall base is at elevation -17.5 m. Upstream the cut-off wall reaches very weathered schist in the central zone and weathered schist near the power house wall, and its base near pier 6E is in the silty sand layer. Downstream, the cut-off wall never reaches the bedrock, being its base in mud and muddy sands (near pier 6E) or in silty sands (near the power house wall). After analyzing the results for a simulation with a perfect cut-off wall and also considering the water levels obtained in boreholes SM1 and SM3, it was concluded that there should be some kind of deficiency at the cut-off wall near borehole SM1. The deficiency in the cut-off wall in the zone of borehole SM1 is simulated by an element of 4.5 m of height and a permeability of 1.3 x 10^{-6} m/s. What's more in the zone of borehole SM1 the initial muddy layer is substituted by sand, to reproduce the conditions found during the execution of this borehole. Finally, the cut-off wall is made to reach bedrock, except in a localized zone, where there is a water flow under the wall through layer 6.

It's also important to refer that, besides the concrete that was found during execution of borehole SM1, a cavity was intercepted under the basin, with a height of around 0.8 m. Under this cavity there was a silty sand layer. Thus, the introduced changes, which made the numerical simulation match the results of the geotechnical study, are realistic and reproduce with some confidence the water flow under basin 7E. Accordingly, the existence of upstream water flow due to wall deficiencies is credible: the first deficiency is localized near the contact of the basin with the alluvial formation and the other near the contact with the bedrock.

The results of simulation b), considering the deficiency in the cut-off wall, are presented in Figure 4. The maximum hydraulic head is 0.44 at the base of the downstream cut-off wall and 0.03 downstream of the basin. Hence, the downstream hydraulic head increases approximately 20% when compared with situation a), with average values. Downstream of the basin the ascending exit hydraulic head didn't register any change.

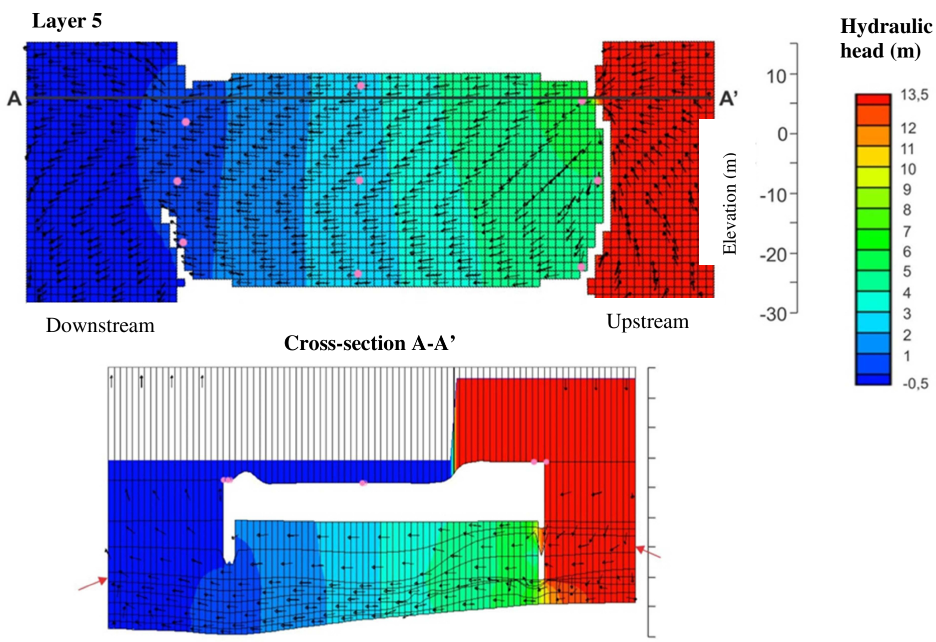


Figure 4 – Results in layer 5 with a reduced sand zone under the cut-off wall, considering its deficiencies and the maximum level upstream and the minimum level downstream of basin 7E [Caldeira et al., 2015].

After implementing the upstream water tightness solution, Figure 5 shows the results of simulation d), considering the maximum water level of the reservoir and the minimum water level downstream. In these conditions, the water flow upstream is now made under the base of the cut-off wall, in the fractured schist layer, where the hydraulic head is maximum. Downstream, the maximum hydraulic head reduces to around 0.05 at the base of the downstream cut-off wall. Moreover, the piezometric level goes down by 7 m, in the region where the upstream cut-off wall contacts with pier 6E.

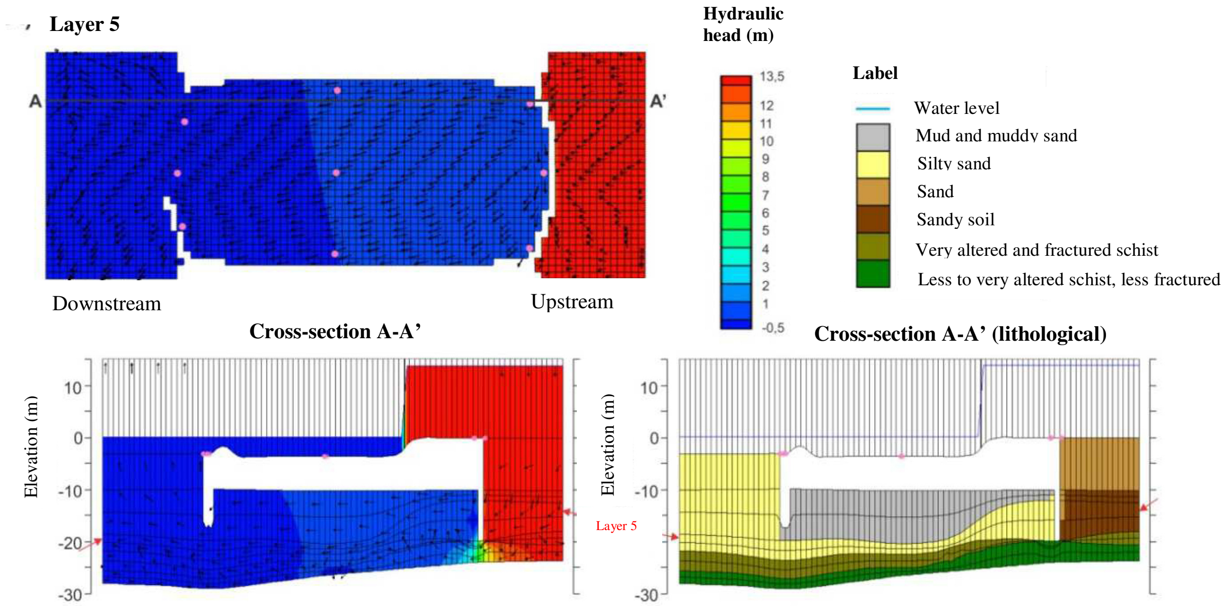


Figure 5 – Results of the numerical simulation in layer 5, after the upstream water tightness solution is implemented, considering the maximum level upstream and the minimum level downstream of basin 7E [Caldeira et al., 2015].

3. Corrective measures implemented in the basin 7E

The analysis of the collected data and of the results of the numerical model led to the definition of corrective measures, in order to improve the safety of the basin. The measures, implemented in 2016, included:

- an upstream water tightness solution, consisting of a secant pile wall, placed upstream of the stilling basin and penetrating the bedrock; the concrete piles were sealed at the top and laterally;
- a foundation strengthening solution using micro piles, in order to minimize basin settlements due to the presence of muddy soils, with very low mechanical strength, as well as pore pressure changes after implementation of the upstream water tightness solution; it should be noted that these changes will lead to an increase of the effective stresses in the alluvial formation;
- a piezometric monitoring system;
- the sealing of the perimeter joint existing between the upstream cut-off wall and the stilling basin.

3.1 Upstream water tightness solution

A plastic concrete curtain of 27 secant piles, with a diameter of 1.50 m and with a distance between axes of 1.05 m, was executed upstream of the existing cut-off wall, below elevation -3.40 m and penetrating at least one meter into the bedrock (Figure 6). The length of the concrete piles ranges between around 14 m and 20 m. The piles were executed from a floating platform, using guide tubes, after the previous rockfill blanket removal above elevation -3.40 m.

The top sealing of the space between the cut-off wall and the curtain of secant piles was performed between elevation -4.60 and -3.40 m by means of a layer of calcium bentonite pellets placed between two layers of plastic concrete, each of them with a 0.40 m thickness. The lateral sealing of the curtain was performed by tubes *à manchette* grouting below elevation -3.40 m and penetrating 5 m into the bedrock. A total of 10 tubes *à manchette* grouting were executed (5 on each side of the curtain).

After the mentioned works, 5 pore water pressure cells were installed, at different depths, in 3 boreholes located upstream of the concrete piles curtain executed in the 7E stilling basin. The depths were selected with the goal of detecting a possible water flow between the pile head and its top sealing and in the interface between the pile base and the bedrock, respectively. The pore water pressure cells installed have been revealing water pressures close to those induced directly by the reservoir water level, corroborating with a suitable behavior of the water tightness solution.

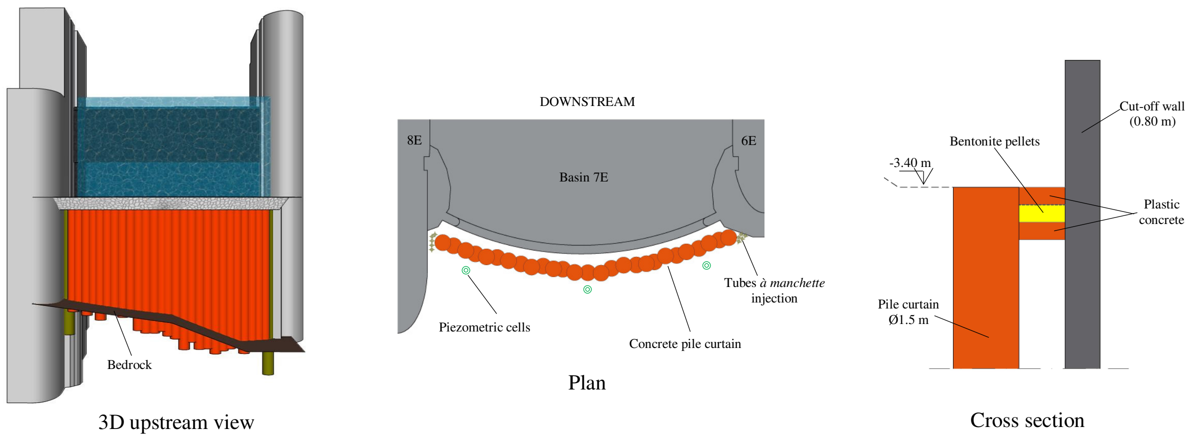


Figure 6 – Upstream water tightness solution.

The corrective measures also included repair works in the perimeter joint between the upstream cut-off wall and the stilling basin, by means of underwater polyurethane injection to prevent the flow previously identified in subaquatic inspections.

It should be noted that this water tightness solution is similar to the previously implemented solution upstream of stilling basins 1E and 3E. However, a different top sealing solution was adapted to reduce execution time when compared to the jet-grouting slab, although with similar effectiveness.

Regarding the quality control of the works, steel tubes were placed in 2 concrete piles of the curtain in order to perform “cross-hole” tests to verify the integrity of the piles. The results of the “cross-hole” tests confirmed an adequate execution of the piles along their length. Concrete slump tests and laboratory tests, such as uniaxial compression tests, were carried out in plastic concrete specimens extracted from the piles. Concerning the lateral sealing of the curtain, during the grouting operations the volume of cement grout and the injection pressures were controlled at each stage of injection and in each valve to satisfy the acceptance criteria. It is worth to note that all these activities have been substantiated by means of diver supervision that, together with bathymetric surveys and underwater inspection and filming, were vital to the quality control of the works, namely the placing of the bentonite pellets and the sealing of the perimeter joint.

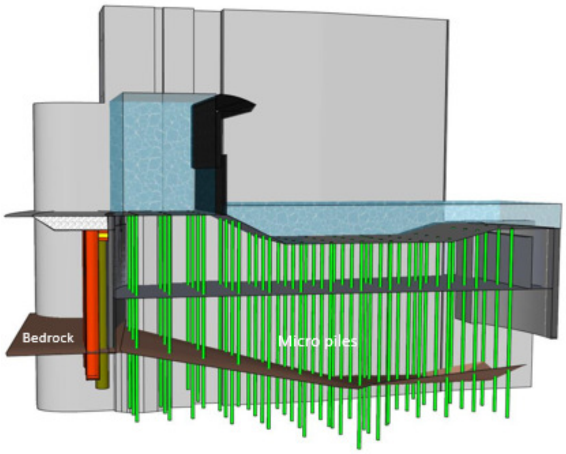
3.2 Foundation strengthening of basin 7E

The foundation strengthening of the stilling basin 7E was carried out by the execution of 97 micro piles (Figure 7), distributed by 13 rows spaced 4 m in the longitudinal direction. In the transversal direction, the micro piles were spaced 3.5 m.

The execution of the micro piles involved the core drilling of the basin, followed by crossing the alluvial formation and, finally, the penetration of at least 6 m in the schist bedrock. The used steel reinforcement was galvanized tube N80, with 88.9 mm diameter and 12 mm thickness, equipped with equally spaced valves (1 m), selectively injectable, along the entire length, enabling the injection of the micro piles. The micro piles were executed through metal tubes (with an inner diameter of 219 mm), sealed in the basin, in order to avoid disturbing the pore pressures under the basin. The top of those tubes was established in accordance with the pore pressures measured during the geotechnical investigation and, globally, does not exceeded elevation 8.00 m.

Two temporary cofferdams have been placed in the stilling basin, one upstream of the gate and another close to the end of the basin, enabling emptying the space between them and core drilling of the basin, as well as sealing of the metal tubes in dry conditions, for 10 rows of micro piles (Figure 7). These micro piles were executed from a fixed working platform installed for that purpose.

The 2 rows of micro piles located upstream of the gate were executed from a floating platform and the one located at the end of the basin using another fixed platform. In these micro piles, installation of the metal tubes was performed by divers. During the micro piles grouting, the volume of cement grout and the injection pressures were controlled at each stage of injection and in each valve to satisfy the acceptance criteria.



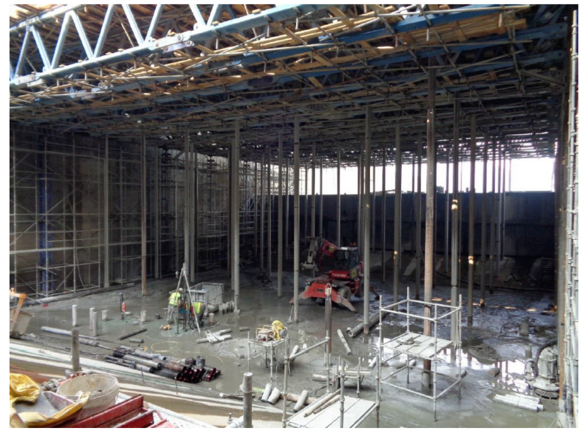


Figure 7 – Three-dimensional representation of basin 7E upstream water tightness solution and foundation strengthening and view of the execution of the micro piles.

5. Conclusions

It can be considered that this recent intervention in basin 7E, similarly those previously executed, namely the intervention upstream basins 1E and 3E, as well as the rockfill blanket downstream the 8 basins, allowed for a significant improvement regarding the Crestuma-Lever dam safety conditions.

The analysis of documentation concerning design and construction of the dam, as well as studies and interventions executed during its operation phase, have led to the conclusion that on basin 7E some specific characteristics could increase the risk of internal erosion and/or undesired movements when compared to the other basins. These characteristics included:

- the geological composition of the alluvial foundation, on which muddy soils with very low SPTs were identified, especially on the topmost 3 m, which could lead to settlement of the basin;
- defects detected during the final construction phase of one of the panels of the upstream cut-off wall, which may have been improperly repaired and could increase the water flow in that zone, consequently leading to internal erosion of the sand layers under the basin;
- the constructive planning, as the basin was built prior to one of its adjacent piers, 8E (wall between the basin and the power plant), whereas the remaining basins were built after both adjacent piers were completed. This planning may have influenced confinement of the soils under the basin and the basin itself during construction, as well as contributed to some voids under the basin near the power plant wall.

The pore water pressure cells recently installed upstream basins 7E, 1E and 3E allow concluding that, after implementing the respective corrective measures, there is no significant seepage through the alluvia foundation of the dam. Moreover, the piezometers installed under basins 5E and 3D have shown no atypical behaviour since their installation, which is positive concerning safety of these basins.

The monitoring of the dam safety conditions will carry on, including the systematic analysis of piezometers and pore pressure cells installed under or upstream the basins, as well as the systematic observation of the downstream rockfill blanket. In order to control seepage conditions in the alluvia foundation of all the basins, additional piezometers will be installed in basins 1D and 5D, which currently do not have this type of equipment.

References

Caldeira L., Miranda L., **Ramos R.,** 2015. "Crestuma-Lever Dam. Analysis of the geotechnical investigation and inspection results of stilling basin 7E. Report 63/2015. Laboratório Nacional de Engenharia Civil, Portugal (*in Portuguese*).

Fernandes I., Lima C., Silva J.D., Queralt M., Caldeira L., Melo J., 2017. “Corrective measures to guarantee the stability of the stilling basins and downstream rockfill in the Crestuma-Lever dam”. HYDROPOWER Congress, Sevilla.

Ribeiro A.A., Lemos J.F, Ramos C.M., 1973. 'Bed protection downstream of a big dam founded in alluvia'. 11th ICOLD Congress, question 41, pp. 689-702, report 38, volume II, Madrid.

Ribeiro A.A., 1976. “Fundamental considerations on the planning of concrete dams founded in alluvia, in large flow rivers. Crestuma dam”. 12th ICOLD Congress, question 46, pp. 265-274, report 17, volume III, Mexico.

Ribeiro A.A., Lemos J.F., Ramalho J., 1979. “Fundamental studies on the planning of construction in large-flow rivers with river bed in alluvia. Crestuma dam”. 13th ICOLD Congress, question 50, pp. 389-402, report 23, volume III, New Delhi.

Ribeiro A.A., Lemos J.F., Cardoso M.M., Neves E.M., 1982. “The exceptional foundations of the gated dams of Crestuma and Coimbra”. 14th ICOLD Congress, question 53, pp. 387-404, report 22, volume II, Rio de Janeiro.

Ribeiro A.A., 1994. “Crestuma-Lever dam. Deterioration of the sand bed downstream the concrete-stilling pools protection”. 18th ICOLD Congress, Q. 71, pp. 221-227, Vol. IV, Durban.

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