RIBEIRADIO DAM FOUNDATION TREATMENT - DESIGN, EFFECTIVENESS CONTROL AND PERFORMANCE

CIGB ICOLD
AUSTRIA 2018 AUTRICHE
26th Congress
86th Annual Meeting
1 - 7 july, Vienna






António MORGADO (*)
Gilberto MONTEIRO (**)
Ricardo OLIVEIRA (*)
Magda QUERALT (**)
Filipa CACILHAS (*)
Celso LIMA (**)

(*) COBA – Consultores de Engenharia e Ambiente SA, Lisboa
(**) EDP – Gestão da Produção de Energia SA, Porto

SUMMARY

The Ribeiradio-Ermida multipurpose hydro scheme, explored since 2015, located in Portugal, encompasses two schemes distanced 5 km. Ribeiradio scheme includes a 83 m high concrete gravity dam and underground powerhouse with 74,5 MW installed capacity.

Geological and geotechnical investigation works revealed a complex geological model at the dam site, featuring great anisotropy as result of intense tectonism to which they were subjected, thus leading to very challenging foundation conditions.

This paper presents the general conception for the dam foundation treatment. Necessary adaptations introduced during the execution stage, to deal with geological, geotechnical and hydrogeological constraints found, essentially associated to geological faults, are also mentioned.

Additionally, this paper describes the methodologies used to assess the effectiveness of the ground treatments, during and after their execution, by means of dedicated verification campaigns, namely water absorption tests and seismic tomography, where the comparison between phases 1 and 2 revealed a significant increase of V_P in the most superficial regions of the rock mass on both banks. Percentage increases of 40-70% in V_P velocities are common in these areas, with a 110% increase obtained on the left bank upper levels. The interpretation of the monitoring data collected so far was also taken into account.

The technical resources used in the control of the first filling of Ribeiradio reservoir proved to be adequate and efficient, allowing the permanent monitoring and interpretation of the dam behaviour. The records of the monitoring data and the results of the visual inspection of the structures didn't reveal any anomalous situations, and they will be important to carry out future studies on the safety conditions of the dam.

RÉSUMÉ

L'aménagement à but multiples de Ribeiradio-Ermida, situé au Portugal et en opération depuis 2015, comprend deux échelons éloignés de 5 km. Celui de Ribeiradio comprend un barrage béton poids à une hauteur de 83 m et une centrale hydroélectrique souterraine avec puissance de 74,5 MW.

Les campagnes de prospection géologique et géotechnique ont contribué à la définition d'un modèle géologique complexe au site du barrage, en révélant une grande anisotropie, conséquence de l'intense tectonisation auquel la région a été soumise, ce qui conduit à des conditions géologiques difficiles.

Cet article présente les caractéristiques générales du traitement de la fondation du barrage et comprend également les adaptations nécessaires introduites en Phase d'Exécution, de façon à faire face aux principales contraintes hydrogéologiques, géologiques et géotechniques trouvées, associées principalement aux failles géotechniques.

En outre, l'article décrit les méthodologies utilisées pour évaluer l'efficacité des traitements du sol réalisés pendant et après leur exécution, à travers des campagnes d'essais de vérification, notamment des essais d'absorption d'eau et de tomographies sismiques. La comparaison des tests réalisés aux phases 1 et 2 a permis de constater une augmentation significative de V_P dans la plupart des régions plus superficielles du massif sur les deux rives. Les augmentations du pourcentage de 40-70% pour les vélocités V_P sont communes dans ces zones, avec une augmentation de 110% obtenue pour les niveaux supérieurs de la rive gauche. L'interprétation des données d'auscultation collectées jusqu'à ce moment a également été prise en compte.

Les ressources techniques utilisées pour le contrôle de la première mise en eau du réservoir de Ribeiradio ont prouvé leur adéquation et efficacité, en permettant le monitoring et interprétation constant du comportement du barrage. L'information obtenue d'après le monitoring et les résultats de l'inspection visuelle des structures n'ont pas démontré de situations anormales et seront importantes pour mener des études futures sur les conditions de sécurité du barrage.

1. INTRODUCTION

The Ribeiradio-Ermida multipurpose hydro scheme, built between 2010 and 2015, is the first major project in the Vouga River basin, in the centre of Portugal. Besides the main dam, Ribeiradio, this project encompasses a second large dam, Ermida, 5 km downstream.

This paper presents the criteria which guided, during Tender Design, the original design of the Ribeiradio dam foundation treatment, as well as the main evolutions underwent to deal with geological, geotechnical and hydrogeological constraints, essentially associated to geological faults, some only identified during construction stage. Furthermore, it describes the methodologies used to assess the effectiveness of the ground treatments, during and after their execution, by means of dedicated verification campaigns, namely water absorption tests, seismic tomography and interpretation of the monitoring data collected so far. Ribeiradio-Ermida Hydroelectric scheme is under commercial operation since the first quarter of 2016.

Fig. 1
Location of Ribeiradio and Ermida hydroelectric schemes
Emplacement des projets hydroélectriques Ribeiradio et Ermida

2. GENERAL DESCRIPTION OF THE PROJECT

Located in the upstream scheme at approximately 4 km east of the village of Sever do Vouga, the Ribeiradio dam created a reservoir with a total capacity of 136 hm^3 at normal water level (NWL) of (110,0), to which corresponds a flooded area of about 561 ha [1, 2].

This concrete gravity dam, the highest of its type in Portugal, has a crest length of 265 m, a 240 m radius circular arch, reaches a maximum height of 83 m, amounting to a total concrete volume of about 300 000 m^3. The dam includes a gated spillway with 2 750 m^3/s design capacity, located over the dam's central blocks, as shown in Figure 2.

This scheme includes, on the left bank, a hydraulic circuit and an underground power house equipped with a vertical axis Francis single unit exhibiting a rated nominal capacity of 74,5 MW, able to turbine up to 125 m^3/s with the reservoir at NWL. The average annual energy generation will reach approximately 117 GWh. The powerhouse layout corresponds to a simple reinforced concrete cylindrical shaft, 22,5 m in diameter, with about 47,0 m maximum depth.

Continuação

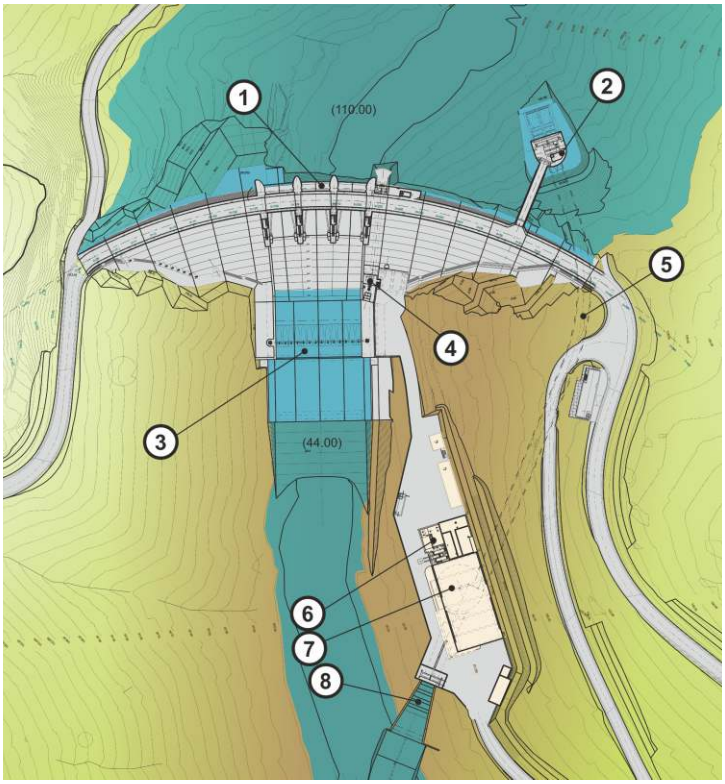


Fig. 2
Ribeiradio scheme plan
Plan du projet hydroélectrique de Ribeiradio

1) Spillway	*1) Evacuateur de crue*
2) Intake	*2) Prise d'eau*
3) Roller-bucket	*3) Bassin à rouleau*
4) Bottom outlet	*4) Vidange de fond*
5) Intake tunnel	*5) Galerie d'amenée*
6) Substation	*6) Poste électrique extérieur*
7) Powerhouse	*7) Usine*
8) Outlet	*8) Galerie de fuite*

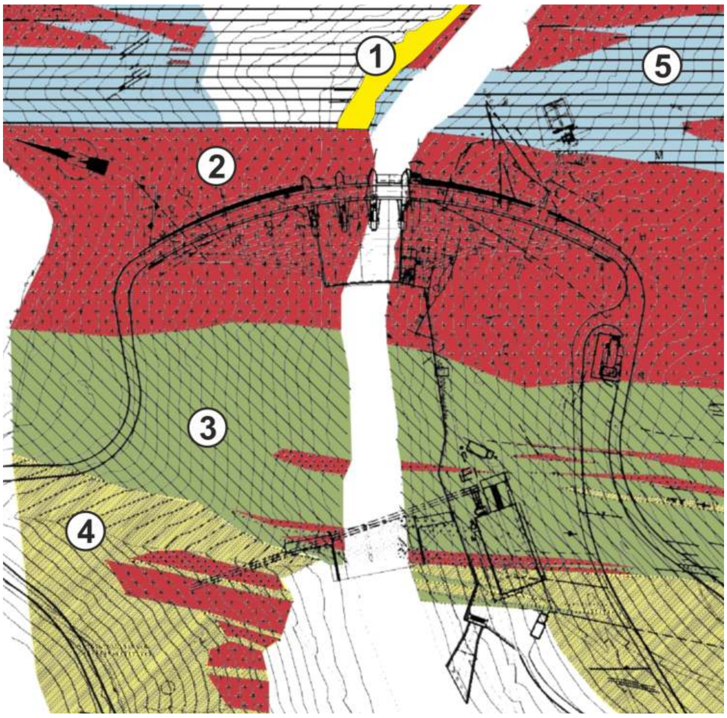


Fig. 3
Ribeiradio geological map
Carte géologique de Ribeiradio

1) Recent – Terrace deposits	*1) Récent – Dépôts de terrasse*
2) Igneous Formations – Granites Late Cambrian:	*2) Formations Ignées – Granites Cambrien Tardif:*
3) Quartzites	*3) Quartzite*
4) Micaschists+Arteritic migmatites	*4) Micaschistes+Migmatites Artéritiques*
5) Micaschists+Venitic migmatites	*5) Micaschistes+Migmatites Vénétiques*

3. RIBEIRADIO DAM FOUNDATION DESIGN AND EXECUTION

3.1. GEOLOGICAL AND GEOTECHNICAL CONDITIONS

At the dam site, the Vouga River runs ENE-WSW through a V shaped narrow valley with steep side slopes and a 30 m wide thalweg. A cross-section of the valley in this area shows that both banks are nearly symmetric, with a 115° subtended angle, the left bank sloping at an average of 30° and the right bank sloping at 35°.

The evaluation of the geological and geotechnical conditions at Ribeiradio's dam site was carried out through a comprehensive investigation program, executed during Tender Design, in order to study the rock mass foundation characteristics. This was done 800 m downstream of a previous site showing very poor rock mass conditions.

Geological and geotechnical investigation works included detailed surface geological mapping, mechanical and geophysical site investigations and in situ and laboratory tests in order to understand the tectonics and metamorphic processes behind the site's rich and complex geotechnical frame. Site investigation works included 38 refraction seismic profiles, 10 seismic fans, 35 boreholes (approximately 1200 m drilled), 5 test adits with a total length of 196 m, 166 Lugeon water absorption tests, 41 dilatometer tests and 2 large flat jack tests executed inside the adits.

Surface mapping showed the dam is set exclusively on an anisotropic hercynian granites band, roughly 100 m wide, which has neighbouring contacts with other geological units: injection migmatites, micaschists and quartzites. The hydraulic circuit crosses all four geological units, shown on Figure 4.

The results also shown the weathering pattern is more intense on the valley's upper levels and especially penetrative above level (90,0) on the left bank. All the formations are heavily tectonized and jointed until significant depths and 10 different joint sets were identified on each bank, mostly subvertical. Main faults, usually corresponding to zones where jointing of the rock mass is more intense and/or where fillings are only a few centimetres thick, were grouped in 6 sets, also mainly subvertical.

By the end of the excavation stage, the results of structural mapping works had revealed 366 faults (101 on the left bank, 265 on the right bank) grouped in 6 main sets – normal faults showing right hand movement – preferably orientated ENE-WSW – which constraint the river alignment on this stretch.

Regarding the rock mass permeability, Lugeon tests showed great absorptions until 20-30 m. At lower levels the tests revealed a progressive reduction in water absorption as the quality of the rock mass increased, generally associated to less intense jointing, until the impervious boundary was reached at depths of 35-45 m on both banks and 20-25 m at the valley's lower levels. Dilatometer tests showed dilatometer modulus below 2 GPa until depths of 30 m; values around 4,5-6 GPa only occur below 25 m, showing a trend towards modulus increase with depth. Large flat jack tests reached deformation modulus of 5 GPa in granites on the right bank (W2-W3, F2-F3), and around 2 GPa on the same lithology on the left bank (W3-W4, F3 to F4-5).

3.2. FOUNDATION TREATMENT – OVERALL DESIGN

Definition of the foundation surface took into account the results from geotechnical zoning and the geomechanical rating of those same zones – both presented in Table 1 [3].

Continuação

Table 1
Foundation rock mass geotechnical zoning
Zonage géotechnique du massif de fondation

Zone	Weathering Degree	Joint Spacing	Absorption (Lugeon Units, LU)	Recovery (%)	RQD (%)	V_l (km/s)	RMR	E (GPa)
GZ3	W_{4-5} - W_5	F_{4-5}	Total loss	< 50	< 25	< 1,0	< 20 – 40	< 1
GZ2	W_3 - W_{3-4}	F_{3-4}	> 10	> 60	40–80	1,0 - 2,7	40 – 60	< 3
GZ1	W_{1-2} - W_{2-3}	F_{1-2} - F_{3-4}	< 3	> 80	> 70	> 3,0	60 – 80	> 5

It was essential to limit the excavation depth to values compatible with the height and type of dam, consequently it was decided that most of the dam foundation surface would be set on GZ2 rock mass showing RMR values between 40 and 60 (fair quality). This design conception led to estimated excavations depths of 5-7 m on the central zone of the valley, 8-10 m along intermediate levels of both banks and 12-15 m above level (90) on both banks (see Figure 4).

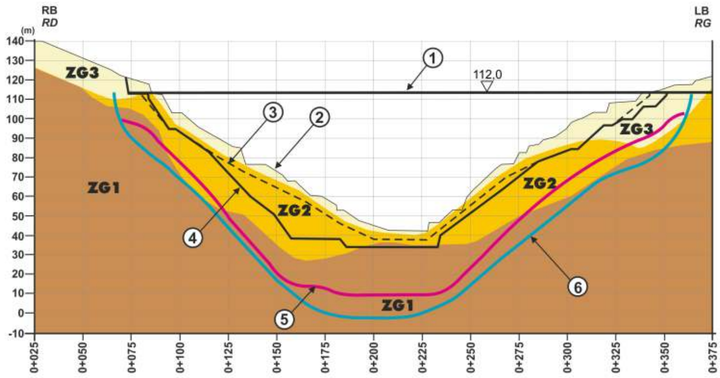


Fig. 4
Geotechnical zoning. Foundation surface. Ground treatment
Zonage géotechnique. Surface de fondation. Traitement de fondation

1) Crest Level	*1) Cote du couronnement*
2) Natural ground	*2) Terrain naturel*
3) Tender Design excavation surface	*3) Surface de fondation de la Phase d'Offre*
4) Optimized excavation surface	*4) Surface de fondation optimisé*
5) Consolidation treatment	*5) Traitement de consolidation*
6) Grout curtain	*6) Rideau d'injection*

In spite of the envisaged excavation, the remaining rock mass at low and intermediate levels of the valley only showed fair conditions and, on the upper levels, the geotechnical conditions were even poorer. This situation called for a systematic consolidation treatment in order to improve the deformability and strength conditions of the foundation and consequently, the dam's foundation operational behaviour. This treatment considered 20-25 m deep grout holes, in order to reach the limit between GZ2 and GZ1 horizons (see Table 1). Hydrogeological criteria were also taken into account in order to ensure consolidation grouting would constitute a primary impermeabilization barrier, given the rock mass underneath the dam foundation was characterized by significant water absorption.

The consolidation treatment aimed at reducing foundation deformability, rendering compatible, as far as possible, both ground and concrete's deformability modulus, promoting an increased shear strength of the rock mass by filling discontinuities (open or filled with soft erodible materials) and improve the sealing conditions at the dam/foundation interface. The efficiency of this treatment greatly benefited from the several joint sets intersections and consequent intercommunication, which promoted a significant increase of the areas subjected to cement grouting.

The consolidation treatment design considered rock mass injections from two differentiated zones: main drainage gallery and downstream drainage gallery (see Figure 5). The consolidation holes were organized in specific treatment sections composed by upstream and downstream fans of inclined boreholes, which allowed injecting systematically with greater density and efficiency the weathered and fractured upper part of the dam foundation.

The improvement of the global behaviour of the dam's foundation rock mass is addressed below, on chapter 4.

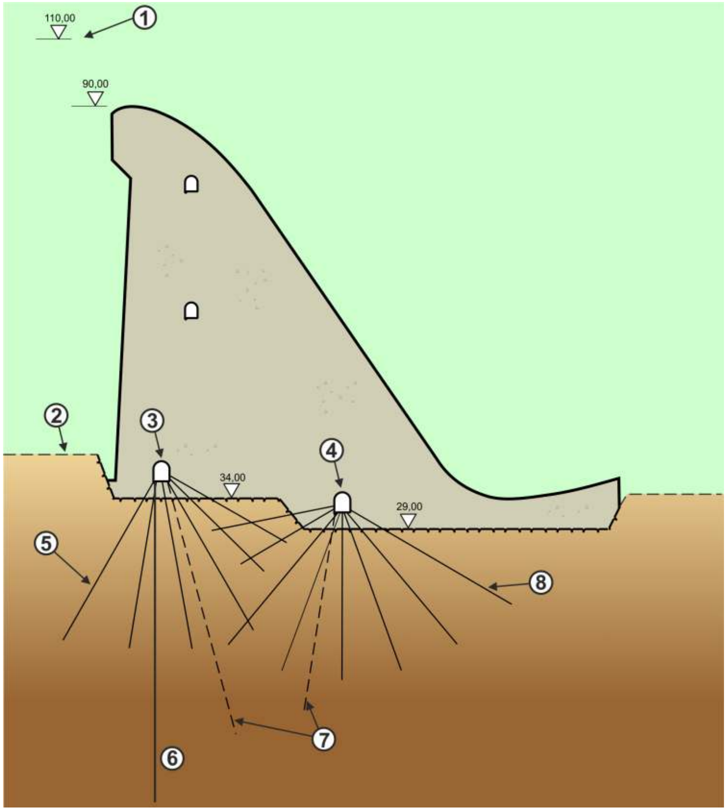


Fig. 5
Foundation surface and rock mass treatment
Surface de fondation et traitement du massif rocheuse

1) NWL=MWL	*1) NNR=NMR*
2) Natural ground	*2) Terrain naturel*
3) Main drainage gallery	*3) Galerie de drainage principal*
4) Downstream drainage gallery	*4) Galerie de drainage d'aval*
5) Consolidation treatment (upstream)	*5) Traitement de consolidation (amont)*
6) Grout curtain	*6) Rideau d'injection*
7) Drainage curtain	*7) Rideau de drainage*
8) Consolidation treatment (downstream)	*8) Traitement de consolidation (aval)*

Owing to the fact that the water tests undertaken inside the boreholes showed occurrences of easy flow at great depths on both banks, a grout treatment was carried out. Given the previous execution of the consolidation treatment, a grout curtain with a single row was considered to be sufficient. Impermeabilization grouting was performed from the main gallery incorporated in the dam and partially from the foundation drainage gallery located in the right bank. On this bank and at the valley's bottom it was estimated that the grout curtain should extend 20 to 25 m below the foundation level, while on the left a similar estimation indicated an extension of 25 to 35 m below the foundation surface.

Two drainage curtains were also foreseen from both drainage galleries – the main, parallel to the upstream surface, along the whole dam length, and the second one, next to the downstream face, only on the spillway blocks. These curtains allows controlling residual seepage below the foundation after the consolidation and waterproofing treatments, but mainly it reduces the installation of uplift stresses at the base of the dam. Tender Design considered also a drainage gallery on each bank at intermediate levels. A cross section of the Tender Design dam's foundation treatment can be seen in Figure 5. The total drilling amounted to 5 300 m.

3.3. GEOTECHNICAL CONSTRAINTS AND DESIGN CHANGES DURING CONSTRUCTION

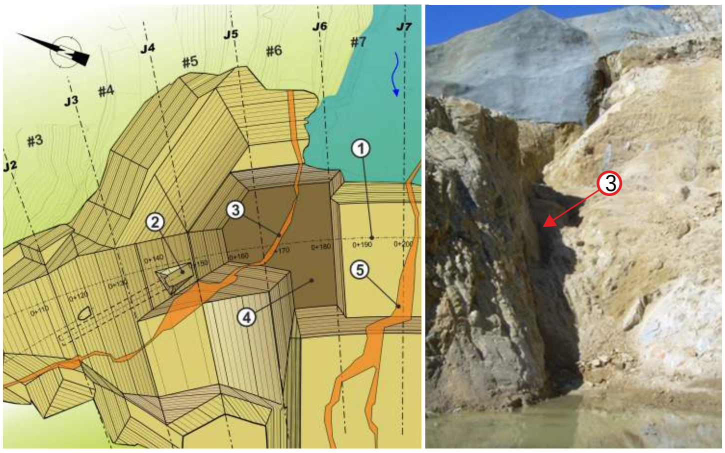
Even if, in general terms, the geological and geotechnical frame found at Ribeiradio's dam foundation agreed with the anticipated geological model, during the excavation works a fault with geotechnical significance was

Continuação

found on the right bank dipping 40-60º towards the river and running obliquely to it, which made its trace follow the dam foundation surface along four dam blocks (see Figure 6). The clayey fault gauge, usually less than 1 m, reached a maximum width of 3,5 m and the intense fracturing and weathering associated increased the heterogeneity already found on the rock mass.

The adequate characterization of this complex fault zone demanded 13 additional cored boreholes. The results showed a significant and abrupt change of the geotechnical conditions between the upstream and downstream sides of the foundation surface, the later exhibiting a moderately weathered (W3) rock mass with moderate to small joint spacing (F3-F4). The upstream half of the foundation surface did not show minimum requirements to set a concrete gravity dam as it was characterized by very weathered to decomposed rock (W4-W5) with small to very small joint spacing (F4-F5). These very asymmetrical conditions were attributed to the combination of lithological heterogeneity, more intense faulting due to the intersection of several minor faults that, together with pegmatite dykes, created suitable circumstances for increased water circulation, thus promoting, locally, much more penetrative weathering until greater depths.

The extent of this situation, not totally foreseen during Tender Design, required a less usual engineering solution contemplating fundamentally, amongst other alterations commented below, an adaptation on the dam shape. This change implied a deeper excavation exclusively on the upstream zone of the foundation, thus creating a step towards downstream in order to ensure adequate foundation conditions (see Figures 6 and 7). By maintaining the geometry of the downstream area it was possible to save a significant amount of excavation and concrete. The structural optimization allowed solving the strength and deformability constraints revealed by the rock mass.



6 a) 6 b)

Fig. 6
Redesigned foundation surface.
Please note additional drainage gallery at right bank's lower levels on 6 a)
Surface de fondation redessinée. Veuillez noter la
galerie de drainage additionnel aux basses cotes de la rive droite dans 6 a)

1) Reference surface	*1) Surface de référence*
2) New drainage gallery	*2) Nouvelle galerie de drainage*
3) Right bank main fault	*3) Faille principal de la rive droite*
4) Over excavation on the upstream side	*4) Surcreusement à l'amont*
5) River bed faults	*5) Failles sous le lit de la rivière*

Due to the clayey nature of the fault filling, which might create an impervious frontier, it was considered of the utmost importance to ensure a more efficient ground drainage on the upstream zone of the foundation resorting to an additional drainage gallery on the right bank lower levels (see Figure 6). This provided an important additional safety element for the dam stability, enabling: a) optimized seepage control on this particular zone of the foundation and b) installation of monitoring equipment, namely, piezometers. It also enables, should it proves necessary, the reinforcement (during operation) of the consolidation treatment and drainage system by means of additional holes.

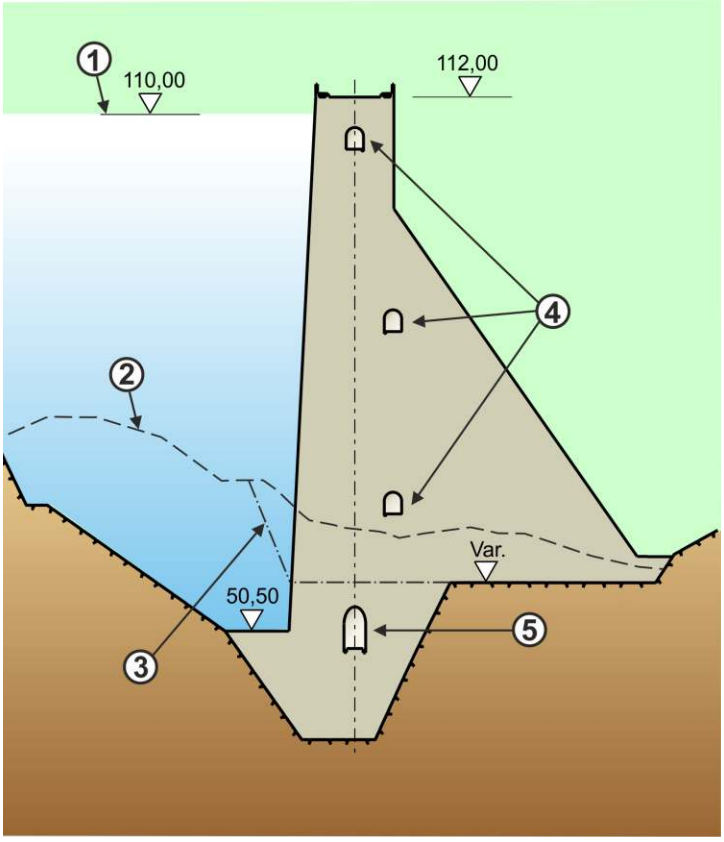


Fig. 7
Redesigned dam cross-section on block 5
Coupe redessinée sur le bloc 5

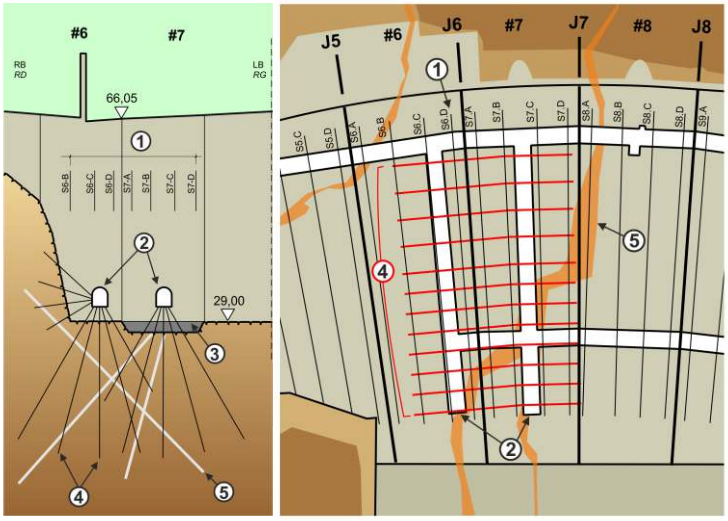
1) NWL=MWL	*1) NNR=NMR*
2) Natural ground	*2) Terrain naturel*
3) Tender Design geometry	*3) Géométrie de la Phase d'Offre*
4) Visit galleries	*4) Galeries de visite*
5) Main drainage gallery	*5) Galerie de drainage principale*

The fault responsible for the river alignment on this stretch was found, as foreseen, on the right side of the riverbed, dipping 40-60°. The fault, 0,5 to 2,3 m in width, showed a filling material including cataclastic granite, quartz and clay. The geotechnical relevance of this fault grew towards the downstream foundation area since there were a combination of several associated minor fault sets responsible for a larger weakness zone.

Since the main compressive and shear forces are directed downstream, special measures were implemented on this zone encompassing 2 to 4 m foundation overdeepening and correspondent replacement for a slab of reinforced concrete. Also a new foundation concrete reinforced beam was placed bellow the downstream foot of the dam, to improve the compatibility of the different blocks' deformation (see Figure 8).

New galleries were also introduced in the dam body to ensure full access for performing an updated version of the consolidation treatment, so that it would be perfectly tailored to the site conditions on this particular zone. Therefore 12 consolidation sections were added – performed from two radial drainage galleries (RG3 and RG5) these sections were normal to the systematic treatment, so to follow and fully intersect the main faults in the river bed – comprehending two sets of five grout holes, disposed in a fan-like manner slopping towards the left and the right bank. The intercrossing of different grout holes orientations (parallel and normal to the river) favoured the foundation treatment's global efficiency, enabling a significant improvement of deformability and strength characteristics to suitable values (see Figure 8).

Continuação



8 a) 8 b)

Fig. 8
Complementary consolidation treatment: a) cross section; b) plan
Traitement de consolidation complémentaire: a) section transversale; b) plan

1) Systematic consolidation sections	*1) Sections de consolidation systématiques*
2) Radial galleries	*2) Galeries radiaux*
3) Reinforced concrete slab	*3) Dalle de béton armé*
4) Consolidation treatment (normal to systematic treatment)	*4) Traitement de consolidation (normal au traitement systématique)*
5) Geological faults	*5) Failles géologiques*

4. EFFECTIVENESS OF GROUND TREATMENT

As mentioned above, Ribeiradio dam foundation treatment comprised consolidation of the rock mass, a grout curtain and a drainage curtain. The effectiveness of these treatments required a control strategy based on: a) results from water absorption tests carried out systematically inside injection holes (from primary to quaternary holes); b) real time analysis of the volume of cement injected in each stage (5 m long), c) seismic tomographies [4] and d) dam monitoring.

The National Laboratory for Civil Engineering (LNEC) performed two campaigns of seismic tomography, the first before and the second after the ground treatment of the rock mass. The first campaign set the reference situation (without ground treatments) and the second one enabled, by comparison, to detect changes in the characteristics and behaviour of the rock mass. The test holes were preserved in order to allow carrying out future measurement campaigns. The monitoring systems installed in the foundations also allowed following its behaviour, with special relevance during the first filling of the reservoirs.

The grout curtain at Ribeiradio dam consisted of a single row of vertical holes, executed from the main drainage gallery, using the split-spacing method. Primary and secondary holes, spaced 6 m and 3 m, respectively, were mandatory. The water absorption tests executed inside tertiary holes (spaced 1,5 m) constituted the first line of approach to the effectiveness of the grout curtain achieved to date. Whenever the results showed water absorptions above 2 Lugeon, new injection holes were open and cement absorption was recorded and compared with values from adjacent holes. Wherever necessary, quaternary holes (spaced 0,75 m) were executed, either for grout injection or for checking the treatment effectiveness on adjacent holes.

Figure 9 shows the cement absorptions recorded inside every 5m grouting stage giving a clear picture of the final grouting density. As shown, despite the unfavourable hydrogeological conditions – with GZ2 typically showing frequent total losses on Lugeon water tests until depths around 25m on both banks during investigation stage – the majority grout take values are low (less than 20 kg/m of grout), with exceptions occurring mainly near geological faults and on the upper levels of both banks.

In fact, the cement absorption values are in line with the recorded water absorptions on the grout curtain primary holes, executed right after consolidating grouting – where total loss evolved to an interval situated between 0 and 15 UL – which comes to prove the important role of consolidation grouting on seepage control.

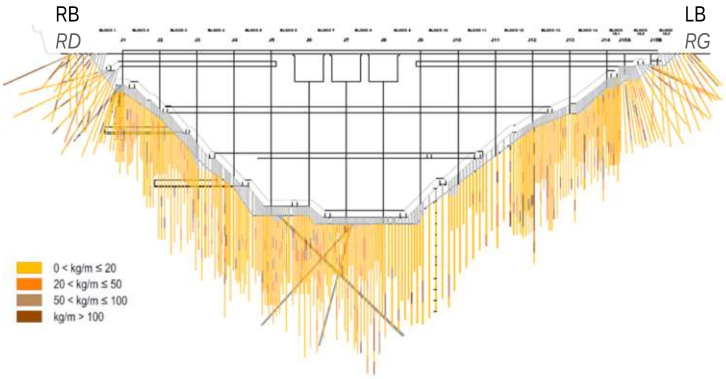


Fig. 9
Grout curtain cement absorptions
Absorptions de ciment dans le rideau d'injection

For the geomechanical characterization of the Ribeiradio dam foundation rock mass crosshole seismic fans were performed before the treatment (phase 1), defining the reference situation, which would be later used comparison with the results obtained after the foundation treatment (phase 2). Test data was interpreted by the seismic tomography method for determining the velocity of the compression seismic waves (V_P) in the various profiles between consecutive holes. The results, showed hereafter, consist in V_P tomographies of the several profiles. The influence of the grouting on the rock mass can be indirectly perceived as the resulting variation in V_P between the two phases, thus allowing to indirectly evaluate the treatment's efficiency [4, 5].

A total of 34 test holes were used at Ribeiradio, 15 in the upstream alignment, 14 in the downstream alignment and 5 in a central alignment, which allowed to analyse 41 profiles between holes. Data acquisition works took place in several campaigns, between November 2013 and February 2015. Figure 10 shows three V_P tomographies of the upstream profile, corresponding, respectively, to phase 1, phase 2, and the V_P percentage variation between phases 1 and 2.

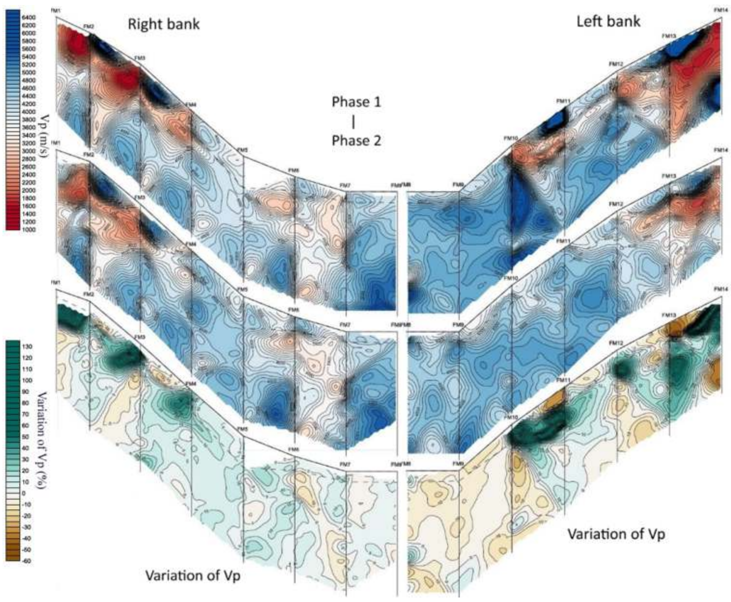


Fig. 10
Seismic tomographies of phases 1 and 2, and V_P percentage variation [4, 5]
Tomographies sismiques des phases 1 et 2, et variation de V_P en pourcentage [4, 5]

Phase 1 results show a relatively heterogeneous rock mass, with V_P distribution lying within a broad range of values: around 1500-2500 m/s on the poor geotechnical zones located in both margins upper to middle zones, but reaching 4500-5500 m/s on the valley's central zone and on the left bank lower levels, corresponding to the higher quality rock mass. There is also a tendency for V_P increase with depth.

Continuação

The comparison between phases 1 and 2 reveals a significant increase of V_P in the most superficial regions of the rock mass on both banks, where phase 1 showed the lowest V_P values [6]. Percentage increases of 40-70% in V_P velocities are common in these areas, with a 110% increase obtained on the left bank upper levels. These results, that are also valid in the downstream profiles, suggest a general and significant improvement on the geomechanical parameters, especially in the rock mass directly beneath the dam foundation, classified under geotechnical zone GZ2.

As expected, due to the higher quality of the massif, there were no significant variations of V_P in the profiles located at the bottom of the valley.

Negative amplitude variations of V_P were also obtained, but are generally of small extent and occur associated, either to zones of strong contrast of speeds, or to the border regions of some profiles, and result mainly from data processing methods.

The total quantities of cement injected and total length of the drainage curtain were as follows:

Consolidation injections (drilling):	19 400 m
Consolidation injections (cement):	706 ton
Grout Curtain (drilling):	12 600 m
Grout Curtain (cement):	266 ton
Total Lugeon Essays (per 5 m/hole):	475 un
Drainage Curtain (Upstream gallery):	4 392 m
Drainage Curtain (Downstream gallery):	908 m

Figure 11 shows the grout take quantities for each dam block as well as the total drilling for the drainage curtain.

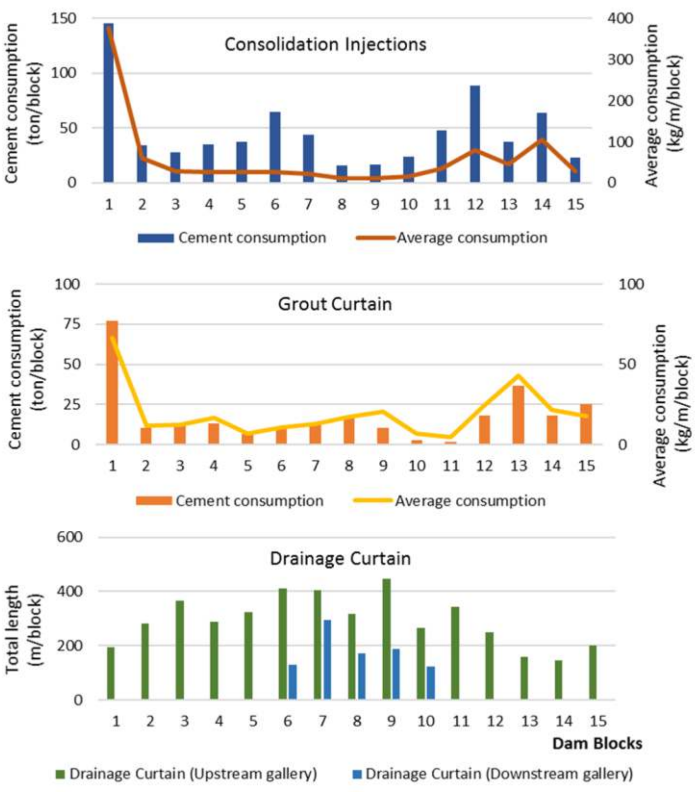


Fig. 11
Main quantities of ground treatment of Ribeiradio dam foundation
Principales quantités de traitement de la fondation du barrage de Ribeiradio

5. RIBEIRADIO DAM FOUNDATION BEHAVIOUR ANALYSIS

The first filling of the reservoir was thoroughly accompanied and analysed in order to access Ribeiradio dam foundation behaviour. For each filling level of the reservoir, and based in the monitoring plan, that considers the measurements of the main loads and of the representative parameters of the structural responses, with the analysis of the behaviour of the dam, the Authority (APA), LNEC, dam´s Owner and design team took the necessary decisions envisaging the continuation of the reservoir filling.

When the last filling level was finished, the foundation treatment was completed but, regarding the observation system, the drainage curtain and the devices for the measurement of total flow rates were still in the final stage of construction. Observation campaigns, visual inspections, geodetic observation and analysis of the water of the reservoir and of the drains foreseen in the plan for first filling of the reservoir, were carried out.

The interpretation of the observed behaviour during the first filling of the reservoir was based on thermal and structural FEM models developed at LNEC (Figure 12).

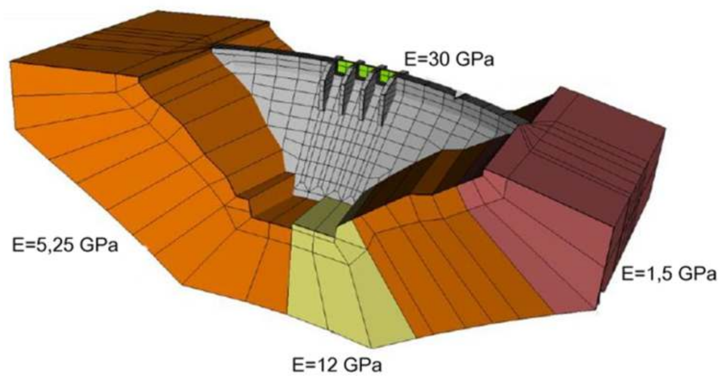


Fig. 12
Finite element model of Ribeiradio dam (LNEC, 2015)
Modèle d'éléments finis du barrage de Ribeiradio (LNEC, 2015)

The temperature distribution inside the dam was estimated using a finite element model which considered the effects associated with air temperature, water temperature, solar radiation and heat of hydration. These temperatures variations together with the hydrostatic pressure of the impounded water were the input of the periodic structural analysis.

The dam construction finished on 15th of September 2014. Since that date, a weekly structural analysis was performed in order to compare their results with the displacements measured by the three plumb lines installed in the dam. Figure 13 illustrates the displacement obtained at the end of the first filling of the reservoir.

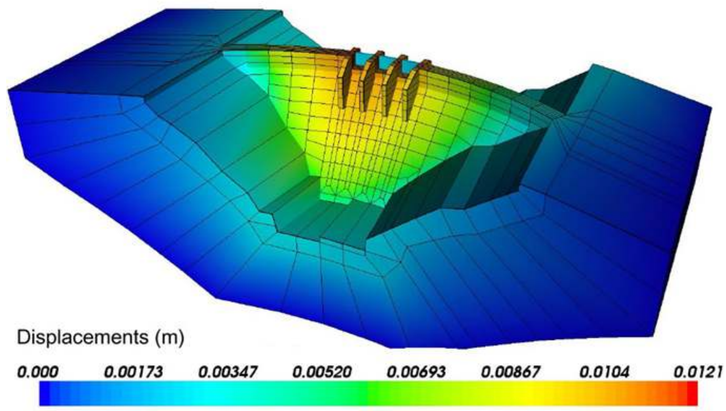
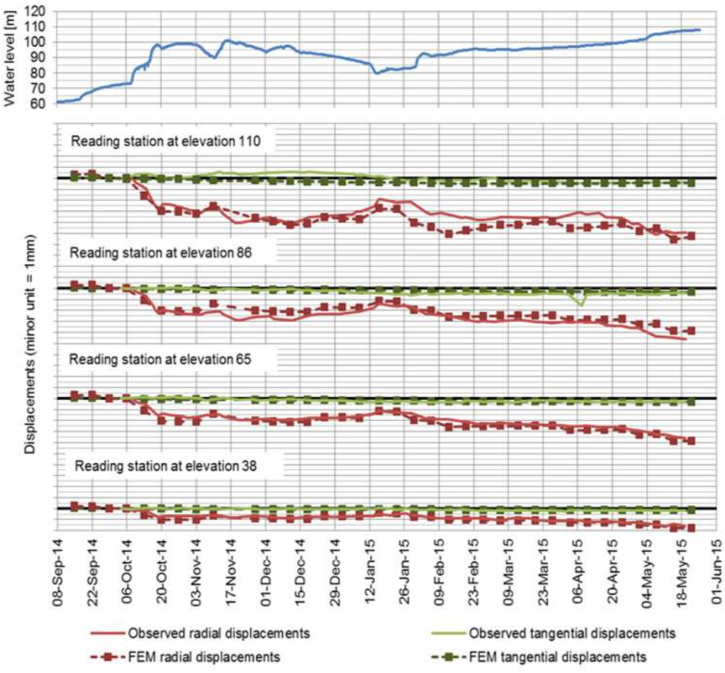


Fig. 13
Finite element model displacements of Ribeiradio dam (LNEC, 2015)
Déplacements du barrage de Ribeiradio (LNEC, 2015)

Figure 14 compares the results obtained in the numerical model simulation with the values of the displacements observed at the different reading stations of the plumb line FPI2. The analysis of this figure shows that, in general, there is a good agreement between the computed results and the observed values. The worse agreement between both values is observed in the higher reading station, and was attributed to this reading station being located in the spillway walls and not in the dam body. Not only the pier is much more flexible than the block, but also, during the considered period, works in the gates of the dam were carried out.

Continuação



(+) Upstream/left bank (–) Downstream/Right bank

Fig. 14
Comparison of calculated and measured radial displacements in the central plumb line FPI 2 (block 8)
Comparaison des déplacements radiaux calculés et mesurés dans la ligne à plomb centrale FPI 2 (bloc 8)

For Ribeiradio dam the displacements evolution observed in the rod extensometers was coherent with the variation of the actions. The water level increase in the reservoir implied, as expected, an increment in the downstream settlement and upward displacement on the upstream zone.

The hydraulic behaviour of the foundation was characterized by reduced flow rates, although high values occurred in some drains, particularly in the bottom blocks of the valley. It was observed that the total drainage flow rates in each block varied between 10 and 15 l/min, except for blocks 6 and 7, where higher flow rates of about 40 l/min were observed in block 6, and approximately 65 l/min in block 7, with a special recommendation being issued for future attention to these blocks in their monitoring of hydraulic behaviour.

Since the drainage system was only fully completed by the end of the first quarter of 2015, the total flow meters could only be observed in early 2016. Figure 15 shows the evolution of the flows by margin and total flows in the dam that were observed during the first year after the end of the filling.

On the blocks located on the right bank, for which the foundation had worst geological and geotechnical characteristics, the flows correspond to about twice of those observed in the left bank. The observed uplift pressures were also small and correspond to percentages of the upstream hydraulic load that never reach 20%.

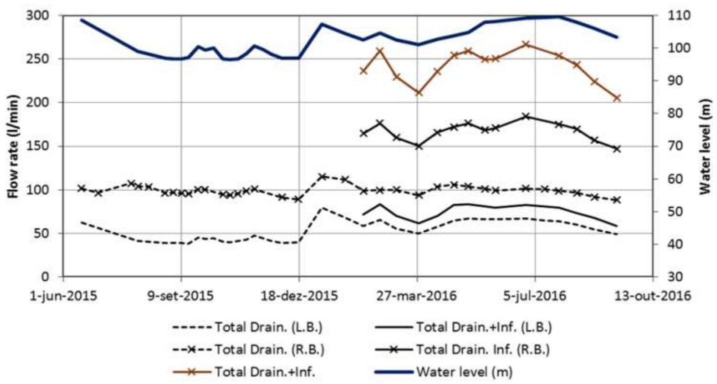


Fig. 15
Flows seepage rates throughout Ribeiradio dam foundation after the first filling
Débit d'infiltration à travers de la fondation du barrage de Ribeiradio après le premier remplissage

6. FINAL REMARKS

Ribeiradio dam site proved to have complex geotechnical conditions mainly due to lithological heterogeneity and anisotropic behaviour because of the intense tectonization. The results from the comprehensive site investigation surveys that have been performed, allowed pointing out most of the geological features and constraints with geotechnical relevance during Tender Design. Still, geological risk is always higher in these scenarios and such geological settings are prone to show new features during the excavation works that demand project redesigning to meet the new site conditions, as occurred in the case of Ribeiradio.

In this framework the success of the Execution Design depended, on one hand, on the designer's capability to accommodate such modifications in real time, since the works were already in progress, which can only be achieved by ensuring good cooperation between all parts involved (in this case, Owner, Designer, Supervision and Contractor). It is also fundamental to ensure that a close and continuous technical assistance is provided. On the other hand, the engineering conception solutions found during early design stages were as flexible as possible in order to accommodate new situations without compromising the general conception of the project.

These good practice international standards made possible to design perfectly tailored engineering solutions to all the geotechnical issues revealed during construction works. The success of this approach becomes evident when a global analysis regarding the effectiveness of the ground treatments, during and after their completion, by means of dedicated verification campaigns, namely seismic tomography, where the comparison between phases 1 and 2 revealed a significant increase of VP in the most superficial regions of the rock mass on both banks. Percentage increases of 40-70% in VP velocities are common in these areas, with a 110% increase obtained on the left bank upper levels.

The structural behaviour of Ribeiradio dam during the first filling of the reservoir was analysed using a thermo-mechanical FEM model, leading to a better understanding of the observed behaviour of the dam. The records of the monitoring data and the results of the visual inspection of the structures didn't reveal any anomalous situations, and will be important to carry out future studies on the safety conditions of the dam. With the integrated analysis of all monitoring data it was possible to understand and evaluate the observed dam behaviour, allowing the control of the evolution of the first filling of Ribeiradio reservoir in a safe mode.

Detailed inspections of the dam and its foundation, as well as the integrated analysis of all monitoring data obtained by the observation systems during the first filling of the reservoir, showed a satisfactory dam behaviour as well as of the respective rock mass foundation. The scheme is currently on its normal period of exploitation for energy production. Careful monitoring of the dam and respective foundation behaviour will prevail over the entire service life the works.

REFERENCES

[1] EDP / COBA - RIBEIRADIO AND ERMIDA HYDROELECTRIC SCHEME. PROJECT, 2008

[2] EDP / COBA - RIBEIRADIO AND ERMIDA HYDROELECTRIC SCHEME. TENDER DESIGN, 2009

[3] RODRIGUES, L., OLIVEIRA, R., SOUSA A. CABRIL DAM - CONTROL OF THE GROUTING EFFECTIVENESS BY GEOPHYSICAL SEISMIC TESTS, 5TH INTERNATIONAL CONGRESS ON ROCK MECHANICS, 1983.

[4] LNEC – CROSSHOLE SEISMIC TOMOGRAPHY AT RIBEIRADIO DAM FOUNDATION. PHASE 1. REPORT, 2015

[5] LNEC - CROSSHOLE SEISMIC TOMOGRAPHY AT RIBEIRADIO DAM FOUNDATION. PHASE 2. REPORT, 2015