Execution of a cofferdam using injections and Jet Grouting columns curtains

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ABSTRACT

The paper presents the study of the implemented and alternative cofferdam solutions for the construction of a complementary spillway on a 50's decade arch dam in Portugal.

The soil improvement and permeability reduction techniques have a wide application in excavation works. The Jet Grouting (JG) technology was adopted to execute a cofferdam that allows a dry and safe area for the complementary spillway construction.

The implemented solution comprised a "L" shape gravity wall and the foundation was insured with injection curtains and JG columns reinforced with steel profiles. Numerical analysis were done using two different 2D finite element models, the SEEP/W to estimate the phreatic level and the PLAXIS 2D the latter performed a stress-deformation analysis, to check the stress state of the various steel resistance profiles modelled, to determine the safety factor on the most important construction phases and finally computed the expected flow to pass underneath the cofferdam in the different cross sections analysed.

Alternative solutions were also presented and analysed, focusing the JG and the Cutter Soil Mixing (CSM) technology to create a continuous percolation barrier to the construction of the spillway safely as well as economically. The water flow in each alternative solution was calculated to estimate the pumping system cost necessary for the continuity of the works later.

At the end of this work it was done a price and cost-benefit analysis for the existing and for possible alternative solutions presented. The analysis of the percolated flow and pricing between all the solutions, JG and CSM shows that, the most competitive technology is the CSM. To conclude the 20% and 30% depth increase CSM solutions seem to be the most interesting ones to be adopted for this case study.

KEY-WORDS: Cofferdams, Jet Grouting, Cutter Soil Mixing, seepage control, hydraulic soil rupture, cost-benefit analysis, 2D FE analysis

1 Introduction

Water percolation through embedded retaining structures generates sometimes considerable hydrodynamic pressures on them. In the past some concrete and earthfill dams have collapsed due to hydraulic rupture of the soils, such as uplift and piping phenomena. In Xanthakos et al,. (1994) it is mentioned that, in a relatively recent past, insufficient considerations in the Jet Grouting (JG) design, with due respect to the drilling and injection process parameters, have contributed to a resistance reduction defined in the project, for a time period less than the defined for the structure.

This paper was elaborated based on cofferdam solution using injection and JG columns curtains to allow Caniçada dam complementary spillway construction, in a dry and safe area. The implemented solution has various restrains, the most notorious of which are the geological and geotechnical scenario, vicinity constraints and the execution time line. With this work was intended to study and evaluate the performance of the most constrained sections of the project, to determine the main excavation phases safety factors and perform the necessary hydraulic soil rupture safety checks present in the Eurocode 7 (EC 7) for each section.

Additionally the water flow to the excavation pit interior for the construction of the complementary spillway, for the same cross sections was analysed. It was also evaluated the cofferdam construction costs and the pumping system costs. This analysis seeks to draw up a cost-benefit relationship chart of the implemented solution and also for the alternative solutions presented.

Each solution was modelled with two 2D finite element software, the first one used was SEEP/W to determine the phreatic level and the second PLAXIS 2D, chosen to analyse the stress-deformation state as well as to quantify the water flow expected for each solution.

2 Case study

The revision of the new Dams Security Regulation (DSR) emerges from the need on the part of EDP Produção to prepare studies with the objective of verifying the fulfilment of the various criteria presented in the DSR document, among them the compliance with spillway capacity for this dam type.

The case study presented in this paper concerns the construction of a cofferdam using injection and Jet Grouting (JG) columns curtains to allow the construction of the Caniçada dam complementary spillway. The dam is located in the parish of Valdozende, county of Sousel, Braga district, at geographical coordinates point 41º 39' 8'' (N) and 8° 14' 5'' (W). The dam operation started in 1955 and the main function of its construction was for electrical energy production, having nowadays other secondary uses, in particular, public water supply and irrigation purposes. This exploitation is composed by two Francis turbines with 62 MW of total installed power, with a maximum fall of 121 m and composed of a concrete dam.

In Figure 2.1 it is possible to see the location and extension of the adopted solution for the cofferdam. This temporary structure had as main function to insure the future excavation works for the complementary spillway construction, in dry conditions, as safely and economically as possible.

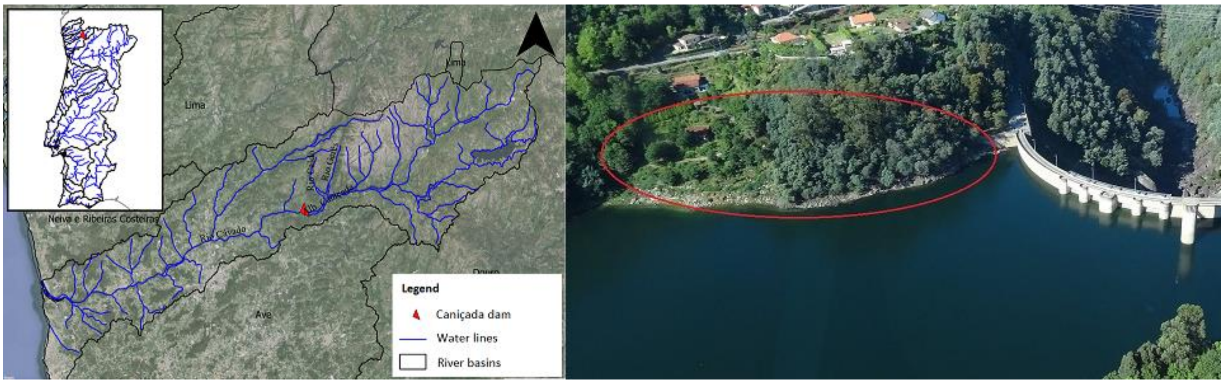

Figure 2.1: Location and delimitation of the complementary spillway construction work site.

2.1 Geologic and geotechnical scenario

Based on the analysis and interpretation of the prospection campaigns contained in the documents Processo de Concurso. Volume V – Elementos de Projecto. Memória Descritiva (EDP, (2012)) and Projecto de Execução. Memória Descritiva e Justificativa - Revisão "A" (Pinto et al., (2014)) were identified and defined four geotechnical zones, ZG4, ZG3, ZG2 and ZG1, resumed in Table 2.1. More information related to this subject can be consulted in (Calatróia, J. G. P. (2016)).

Table 2.1: Estimated values for the geomechanical parameters (adapted from Pinto et al., (2014)).

Geotechnical zone	Typical description	N_{SPT} Caract.	γ (kN/m^3)	ϕ' (°)	c' (kPa)	E (MPa)	$k_x=k_y$ (m/s)
ZG4	Fluvial beach sands and released deposit material	$1 \leq N_{SPT} \leq 10$	18	30	-	10	$1x10^{-5}$
ZG3	Granite residual soil to granitic decomposed massif	$10 \leq N_{SPT} \leq 60$	19	38	5 -30	50	$1x10^{-5}$
ZG2	Granitic rocky massif	$50 \leq$ RQD (%) ≤ 75	20	40	100	100	$1x10^{-5}$
ZG1	Granitic rocky massif	$75 \leq$ RQD (%) ≤ 100	21	40	300	400	$1x10^{-5}$

2.2 Vicinity constraints

The construction of the cofferdam was held in the Caniçada dam basin, and the phasing constructive work was significantly conditioned by the change of the basin water levels in each work period, as well as by the need to prevent the occurrence of any spillages of slurry grout to the basin.

2.3 Execution time

The productivity of all the works associated with the implemented solution using the JG technique should ensure the deadlines for the construction of the complementary spillway, in safe and in economy conditions for the work and to the surrounding structures and existing infrastructures. In particular there was the need that the cofferdam works would be finish by the end of September 2014.

2.4 Executed solution

The executed solution of the cofferdam consists essentially, in a "L" shape top gravity concrete wall in order to resist the water pressure from the basin. The wall foundation can be divided in two, since the micropiles were embed in different geotechnical zones along the longitudinal cofferdam dimension. The first section, located on the left abutment and in the mid-section of the cofferdam is founded by a double overlapped ∅1000 mm diameter JG columns with N80 hollow tubes spaced from 0,80 m apart. The intention was to create a vertical waterproof cut-off wall and ensure a proper vertical stability. HEB 140 steel profiles and 30° inclined HEB 160 steel profiles were included with a 1,60 m spacing (see Figure 2.2) to accommodate the slide and overturning forces generated from the water level next to the wall, transferring them to competent layers. The other section located on the right side of the cofferdam, next to Caniçada dam, is where the cofferdam foundation intersects granitic rocky massif. The foundation has one set N80 steel profile plus one ϕ25mm rebar, at the front and the rear of the wall to resist the overturning force (see Figure 2.3).

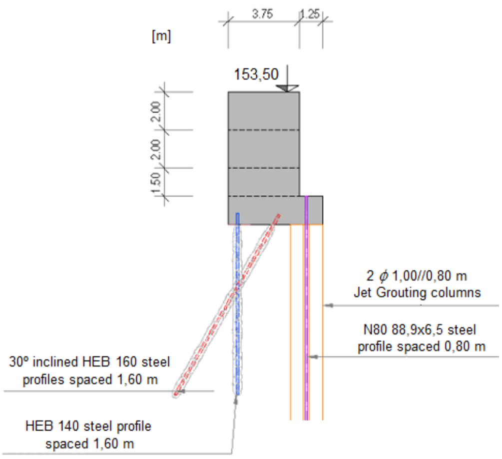

Figure 2.2: Wall geometry and foundation solution adopted for the left abutment and middle section of the cofferdam (adapted from Pinto et al., (2014)).

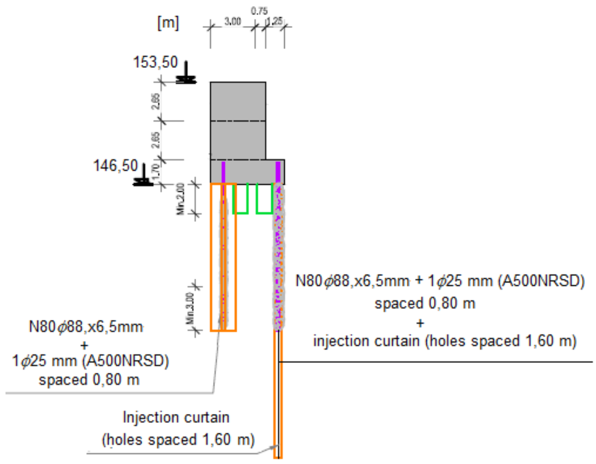

Figure 2.3: Wall geometry and foundation solution adopted for right abutment of the cofferdam (adapted from Pinto et al., (2014)).

2.5 Monitoring plan

The Monitoring Plan (MP) constitutes a tool for prevention and risk management, aiming to ensure the realization of all works in conditions of safety and economy, as well as the analysis of the surrounding structures and infrastructures behaviour. The modelling results were confirmed with the MP records so the hypotheses considered initial could be confirmed.

2.6 Alert and alarm criteria

The alert and alarm criteria (see Table 2.2) were established based on the type of solution for the interventions to be implemented, as well as the geology of the site and the model calculations results presented in the document Projecto de Execução. Cálculos Justificativos - Revisão “B” (Pinto et al., (2014).

Table 2.2: Alert and alarm criteria (adapted from Pinto et al., (2014)).

	Alert criteria			Alarm criteria		
Phase	Vertical displac. (mm)	Horizontal displac. (mm)	Flow (m^3/day)	Vertical displac. (mm)	Horizontal displac. (mm)	Flow (m^3/day)
Water level raising	3	5	50	6	10	100
1st excavation level	4	10	100	8	15	200
2nd excavation level	5	15	150	10	20	300
3rd excavation level	5	20	300	10	30	600
4th excavation level	7	30	500	14	40	1000
5th excavation level	8	40	700	16	60	1400
6th excavation level	10	50	900	20	70	1800

3 Modelling of the existing solution

2D finite element software (FEM) were used to model three different cross sections that present more challenges at the design phase of the executed solution for the cofferdam. This modelling aims to compare the numerical program results with the monitoring records from the work site. Other objectives of the modelling were to perform the necessary safety checks for hydraulic soil rupture, according to EC 7, that determines the safety factors for the main construction phases, quantifies the expected flow into the excavation pit and finally puts alternative solutions, so that a chart cost-benefit for all the solutions presented can be elaborated. More information related to this subject can be consulted in (Calatróia, J. G. P. (2016)).

3.1 SEEP/W

The hydraulic conductivity parameters necessary for the SEEP/W modelling to each geotechnical zone (see Table 3.1), defined in chapter 2.1, were consulted from the documents Processo de Concurso. Volume V – Elementos de Projecto (Peças Escritas). Memória Descritiva (EDP, (2012)) and Projecto de Execução. Cálculos Justificativos – Revisão “B” (Pinto et al., (2014)). This analysis was performed using the “Saturated only” model and SEEP/W modelling results can be visualized in Figure 3.1.

Table 3.1: SEEP/W soil model parameters.

	Earthfill	ZG4A	ZG4B	ZG3	ZG2	ZG1	JG
k_x=k_y (m/s)	$1x10^{-5}$	$1x10^{-5}$	$1x10^{-5}$	$1x10^{-5}$	$1x10^{-5}$	$1x10^{-5}$	$1x10^{-8}$

<u>**Note**</u>: It is necessary to mention that from the provided documents that support this work, it’s not possible to estimate the granitic rocky massif permeability, defined as ZG2 and ZG1. Table 3.1 presents the four

geotechnical zones (ZG4 to ZG1) as isotropic and homogeneous, regarding the permeability coefficients in x and y directions ($k_x=k_y$). This assumption may not be within the safety side, since it could not model the accurate behaviour near the geotechnical zones boundaries.

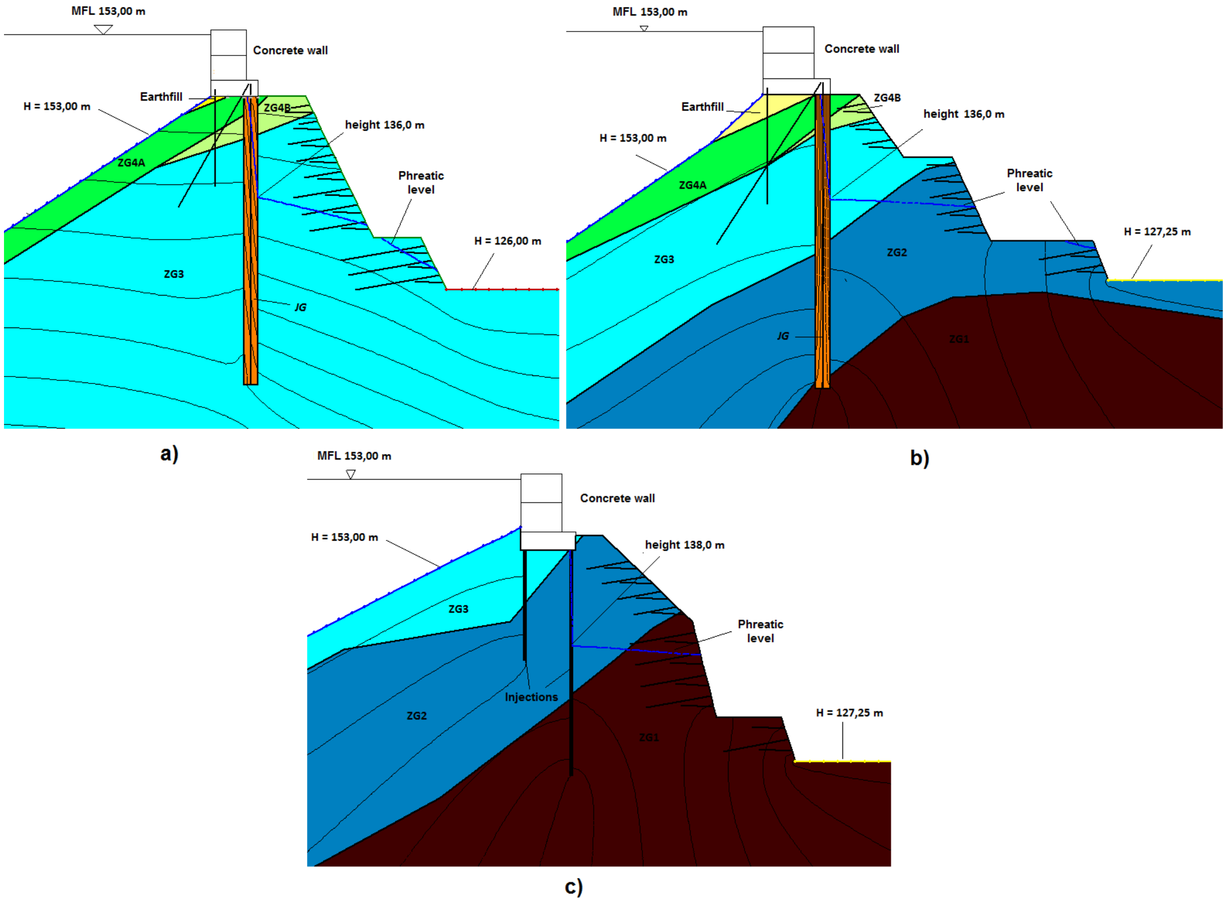

Figure 3.1: SEEP/W phreatic level calculation: a) Section 7, b) Section 8 and c) Section 10.

3.1.1 Modelling and piezometers records comparison

Comparing the piezometers records from the site and the results from SEEP/W it was possible to validate the results, since the boundary conditions defined for the modelling (in particular the MFL (153,00m), and the observed basin water level, 151,00m height) it was observed that the difference of the phreatic level was approximately only 1 meter apart.

3.2 PLAXIS 2D

The second software used in this work was the PLAXIS 2D in order to study the bidimensional stress-deformation state of the three chosen cross sections. With this software it was possible to achieve the remaining objectives defined for the work, calculating the safety factors of the main construction phases, perform the safety checks already mentioned and quantify the flow for each chosen section.

First it was necessary to define the soil model to be used that could replicate a proper soil behaviour. The "Hardening Soil" model was chosen since this can replicate more accurately the load and unload soil response on excavations construction phases (Raposo, N. P. (2007)). In Table 3.2 are the material parameters and the parameters used to characterize the soils at Table 3.3.

Table 3.2: PLAXIS 2D materials parameters.

Material	γ (kN/m^3)	ϕ'(°)	c´ (kPa)	E (GPa)	k_x=k_y (m/s)
Concrete	24	Linear elastic		31	-
JG columns	21	38	180	1	1x10^{-8}

Table 3.3: PLAXIS 2D soil parameters.

Parameters	Earthfill	ZG4A	ZG4B	ZG3	ZG2	ZG1
γ_{unsat} (kN/m^3)	18	18	18	19	20	21
γ_{sat} (kN/m^3)	19	19	19	20	21	22
k_x=k_y (m/s)	1x10^{-5}	1x10^{-5}	1x10^{-5}	1x10^{-5}	1x10^{-5}	1x10^{-5}
E_{50}^{ref} (kN/m^2)	10000	10000	10000	50000	100000	400000
E_{oed}^{ref} (kN/m^2)	10000	10000	10000	50000	100000	400000
E_{ur}^{ref} (kN/m^2)	30000	30000	30000	150000	300000	1200000
c´ (kN/m^2)	2	2	2	20	100	300
ϕ'(°)	30	30	30	38	40	40
m (-)	0,50	0,50	0,50	0,50	0,50	0,50
Ψ (°)	0	0	0	0	0	0
Material type	Drained	Drained	Drained	Drained	Drained	Drained
Interface reduction factor			Rigid			

3.2.1 Safety factors – phi-c reduction

The phi-c reduction procedure available on the PLAXIS 2D was used to determine the safety factor values of several constructive phases. The obtained values for the three studied sections can be seen on Table 3.4.

Table 3.4: Safety factors obtained through phi-c reduction procedure for the most import construction phases.

	After wall concreting	Water level raised to 153,0 m	1st excavation stage	2nd excavation stage	3rd excavation stage
Section 7	1,86	1,63	1,43	1,33	-
Section 8	2,17	1,91	1,52	1,34	1,28
Section 10	3,19	3,05	2,67	2,03	-

3.2.2 Modelling and monitoring results comparison

Concluded the numerical analyses with the software it was necessary to compare the modelling values and the monitoring records. The horizontal displacements to the excavation interior are summarized in Table 3.5.

Table 3.5: Horizontal displacements from modelling and from the monitoring devices at work site.

	Modelling	Inclinometers
Section 7	44,1 mm	40,0 mm
Section 8	30,9 mm	10,0 mm
Section 10	15,5 mm	22,0 mm

Considering the data in Table 3.5 the horizontal displacements prediction of the model is considered to be valid for the three sections, proving the numerical analyses with this software adequate and that it is a good tool for geotechnical structures design. Regarding the horizontal displacement difference observed for the Section 8, this can be explained due to a new inclinometer installation after 10 months of the beginning of the excavation works, which certainly would be higher than the recorded.

In order to estimate the total flow expected to the excavation pit it was necessary to consider an additional section, Section 2. The choice criteria for this section were the reduced length of the JG columns held at ZG3 layer and also the confirmation at the work site of a higher water flow for this area then expected on design phase. The analysis of this Section 2 could delimit the influence areas for each section and determine the total flow expected to percolate to the interior of the cofferdam. The total flow obtained through modelling and the record from the work site, can be seen in Table 3.6.

Table 3.6: Flow values from modelling and from work site records.

	Modelling	Registered on the work site
Water flow to the excavation pit	678 m^3/day	6480 m^3/day

The explanation for the disparity, of the values can be in part due to the shallow JG columns depth next to the left cofferdam abutment and to the need to extend longitudinally the cofferdam closing it at a ZG2 (granitic rocky massif). The performed modelling for the Section 7, Section 8 and Section 10 was considered to be appropriate since it was confirmed on the work site a reduced water flow on those areas. The additional Section 2 analyses is insufficient to model the real percolation behaviour to inside the cofferdam, since the water bypasses laterally and underneath existing curtain. More information can be consulted on Calatróia, J. G. P. (2016).

3.2.3 Hydraulic desestability and safety checks

In the particular case of this thesis, given the geological-geotechnical existing scenario the hydraulic desestability safety checks needed to be performed, according to EC 7, are the hydraulic heave and piping phenomenas.

3.2.3.1 Piping

The piping safety check was performed only for the Section 7 since the bottom of the excavation pit represents ZG3 composed by granite residual soil to granitic decomposed massif. The last excavation level for the Section 8 (ZG2) and Section 10 (ZG1) were defined as granite rocky massif and the piping didn't occur for rocks masses. The section Corte 7 didn't verify safety, with a safety factor equal to 0,48.

More information related to the piping safety check can be consulted in Calatróia, J. G. P (2016).

3.2.3.2 Hydraulic heave

The safety check of the hydraulic heave was performed for the most unfavourable constructive phase, corresponding to the last excavation level modelling of each section, since hydraulic potential between the Caniçada basin at MFL (153,00 m) and the interior excavation is maximum. In each section were chosen multiple points where the vertical velocity vectors were higher, between the JG columns and the excavation slopes face. The action force values were increased by factor $\gamma_{Q;sup}$ equal to 1,50 and the resistance forces decrease by factor $\gamma_{G;inf}$ equal to 0,90, according to EC 7. The safety condition was checked for all three sections studied and can be consulted in Calatróia, J. G. P (2016).

4 Modelling of the alternative solutions

The modelling of the alternative solutions was predesign only for the section where a geological and geotechnical scenario was more unfavourable and with larger displacements occured. It was initially tried to optimize the wall geometry, reducing the constructing costs but it wasn't possible, since it didn't verify the slide and overturning safety checks presented in EC 7.

The alternative solutions can be divided in three parts: (i) using the same JG technology that has been used for the executed solution, increase the columns depth in 10%, 20% and 30% and quantify the expected flow in each one; (ii) to present a new solution using the CSM technique. This last one can create a continuous percolation barrier with less joints comparing to the JG solution, analysing the displacements, safety factors, safety checks for hydraulic soil rupture according EC 7, and determine the water flow inside the cofferdam; (iii) to repeat the process defined at (i) increasing in the same percentages (10%, 20% and 30%) the panel depth and quantify the expected flow into the excavation pit.

4.1 Alternative solutions using JG and CSM techniques

Another objective defined for this work was to present alternative solutions that safely and economically were equivalent to the executed cofferdam solution for the complementary spillway construction. Only the Section 7 was studied, since this one presents the higher displacements and a geologic-geotechnical scenario more unfavourable than the other sections Corte analysed before. Initially it was performed a wall geometry change analyses to reduce materials and construction costs, but they didn't check the safety to slide and to topple actions, according to EC 7. The geometry of the excavation slope was unchanged due to the vicinity constraints of the complementary spillway construction.

The alternative solutions were divided in two. One with the same technology as the executed solution, increasing the JG columns length in depth, and a second where the Deep Soil Mixing methodology was studied, selecting the Cutter Soil Mixing to create a continuous cut-off wall alternative solution considering the existing constraints and being an economic viability to the executed JG solution. The alternative solutions using JG columns were defined with 10%, 20% and 30% length increased and the expected flow for the interior of the excavation pit was quantified. The safety of this alternative solutions is theoretically assured with the columns depth increase, so there was no need to perform all the safety checks done previously. On this analyses only the expected flow for each alternative JG solution was quantified.

The second part of the study of alternative solutions is focused on the CSM technique and it was necessary to perform the same analyses as for the three sections presented previously in chapter 3. The wall geometry and the concrete class are the same as shown in Figure 2.2, but the spacing of both micropiles HEB 140 (vertical) and HEB 160 (30º inclined) was reduced to 1,5 m to construction purposes and the N80 steel profile removed since CSM technique doesn't need the tubular profile to execute the soil panel. A wall thickness of 0,60 m was defined to be a common choice for this type of structures. The main results for the alternative solutions in JG and in CSM can be consulted in Calatróia, J. G. P (2016).

4.2 Cost-benefit analysis

Estimated the total construction cost and the pumping system cost in €/lm, it is now possible to evaluate the cost-benefit ratio of each presented solution, and highlight their advantages and disadvantages. In Figure 4.1 the diagram represents on the x axis the total construction cost and in the y axis the pumping system cost accounting to the flow expected for each solution. The operating time for the pumping system was defined for 22 months (or 660 days) considering to the project schedule for the complementary spillway construction.

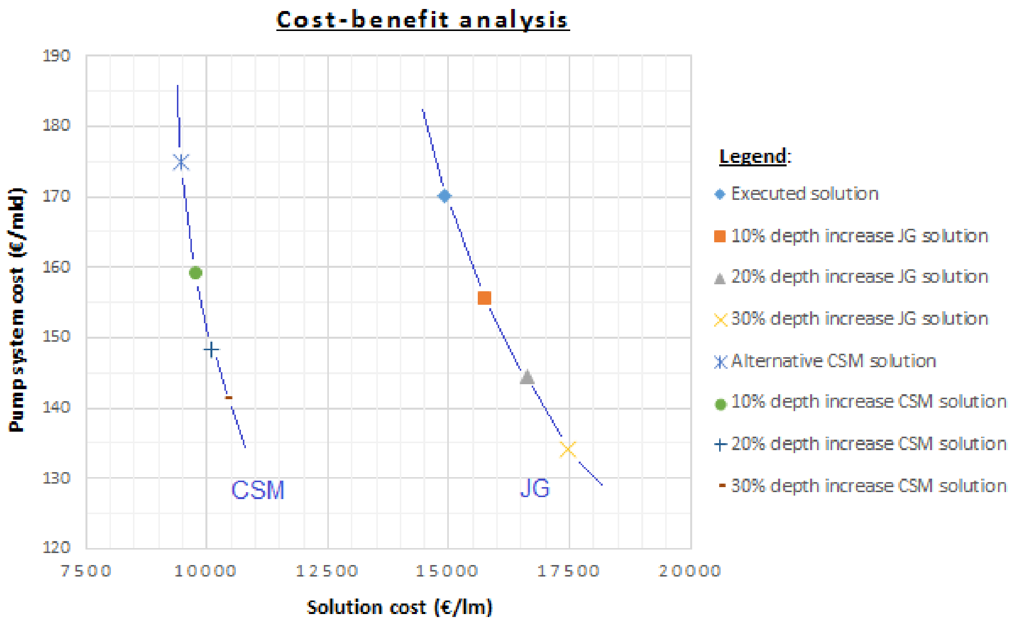

Figure 4.1: Cost-benefit chart for each solution presented.

As can be observed in Figure 4.1, the CSM technology in comparison with the JG technology is more attractive. The cost among the CSM solutions don't vary significantly, being the difference between the alternative solution in CSM and the 30% depth increase in CSM of 915 €/lm. The cost difference for the pumping system between them is 34€/lm. For the JG technology the cost difference between the implemented solution and the 30% depth increase in JG is more significant, with a value of 2546 €/lm, around 2,8 times more expensive and with a pumping cost difference of 36 €/lm.

Another aspect from the diagram analysis is the more accentuated decrease that the JG technology allows with the same length increasing proposed for the alternative solutions. In comparison with the CSM technology the flow reduction function with the curtain length is lower.

Finally it is relevant to refer that from all the presented solutions, the 20% and 30% depth increase solutions made from CSM cement-soil panel seem to be the most interesting from the point of view of their cost-benefit ratio. Nevertheless these solutions do not check the safety against piping phenomena and it is expected the necessity to adopt protective filters and riprap placing, ensuring the reduction of the maximum hydraulic gradient to values below the critical soil hydraulic gradient corresponding to the last excavation level existing soil.

5 Main conclusions

The main objectives of this work were to study the behaviour of the executed cofferdam solution with JG technique for the construction of the Caniçada dam complementary spillway; to perform the EC 7 safety checks regarding the soil hydraulic rupture; to present alternative solutions and proceed to a cost-benefit analysis of all the solutions.

The SEEP/W software has proven to be a powerful and robust tool to analyse the water flow through soils, registering a difference of 1 m, between the modelling values and the monitoring records. The modelling performed with PLAXIS 2D software placed more challenges to the analysis of the chosen sections, requiring more inputs and the need to choose which soil models are more suitable for the soil and structural components modelling. Concluded the modelling, the results were validated and have been considered satisfactory for the expected behaviour of the cofferdam.

In relation to the expected flow into the excavation pit, it was necessary to analyse an additional cross section, Section 2, located near the left cofferdam abutment, allowing the influence areas delimitation and the quantification of the total water flow through the executed cofferdam. Direct conclusions were not possible to be made since the modelling values and the monitoring records were very divergent, registering a higher water flow on the construction site, then the one expected at design phase, exceeding the alarm criteria values defined for the project. To this date the project is still being re-designed to allow the remaining works to be done in safe and dry conditions, making a significant delay of all Caniçada dam complementary spillway project.

As possible causes for the differences observed stand out the reduced length of the JG columns near the left abutment, less 14 m and 21 m columns length embed in the same geotechnical zone, ZG3, comparing to the rest of the cofferdam, with the exception with the right abutment embed in granitic rocky massif. It is also important to mention the geological-geotechnical scenario complexity, with a very heterogeneous soil and the existence of numerous granite blocks (boulders) in depth, presenting significant challenges to the execution of a continuous JG and injections cut-off wall.

Another objective for this work was to perform the necessary safety checks, according EC 7, for the hydraulic soil rupture of the executed solution. In relation to the piping phenomenon, the cross Section 7 didn't verify the safety, with a safety factor equal to 0,48. The geometry conditions for the construction of the complementary spillway didn't allow a new slope configuration for the alternative solutions and the increased depth percentages (10%, 20% and 30%) are insufficient to reduce the hydraulic gradient of the ZG3 soil to values below is critical soil gradient. This concludes the need to implement protective filters and riprap material with high permeability coefficient in this cross section, protecting the slope and avoiding a global failure mechanism to be formed and to close the left cofferdam abutment in a proper geotechnical zone (ZG2).

Quantified the water flow to the interior of the excavation pit and pricing for each presented solution in JG and in CSM, it was concluded that the most competitive technology seems to be the CSM, being the 20% and the 30% length increase the most competitive among the remaining solutions. These two solutions, correspond to a 12,9% and 17,0% flow decrease comparing to the executed solution in JG. It is also relevant to mention that the 30% length increase JG solution is the one that presents a greater reduction of the percolated flow, 21% less compared with the executed solution.

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