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Geotechnical behavior of gravity dams built on sedimentary rocks: pore pressures and deformations analysis of Dona Francisca HPP foundation

Verlei Oliveira dos Santos[1#], Luiz Antônio Bressani[2],
Camila de Souza Dahm Smirdele[3]

Article

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Abstract

In Brazil, some dams have been built on sedimentary rock masses, which usually present greater deformability and permeability in comparison to metamorphic or igneous rock masses. This article describes a case study whose goal is to present and analyze the main data related to the monitoring of foundation behavior of the Dona Francisca dam, whose foundation is essentially constituted by sedimentary rocks. Dona Francisca gravity dam is a hydroelectric power plant (HPP) and was built in 2000, on the Jacuí River, in the central region of the Rio Grande do Sul State, Brazil. The analysis of the foundation behavior was done in terms of pore pressures and deformations recorded during seventeen years of dam operation. The geological and geotechnical conditions of the foundation are related to the Formação Caturrita rocks, made up mainly of sandstones and intercalated levels of siltstone and argillite. In the first five years of operation there was an intense stabilization process of the foundation rocky mass. After this period, it was verified the occurrence of stabilization at a lower rate. The deformation of the Dona Francisca HPP foundation is higher when compared with other larger dams, such as the Itaipu HPP dam. It was carried out an analysis of the 18 vibrating wire piezometers data, allowing a global assessment about the uplift water pressures behavior. Most piezometers indicated a reduction in the pore pressure values over time with a current trend of stabilization, and readings below the control values recommended in design.

1. Introduction

Kanji et al. (2020) drew attention to the investigations on soft rocks are important mainly due to the fact that in designing important workings involving soft rocks, their properties are difficult to establish and most of the times their parameters are adopted on the conservative side, against the economy of the project. The definition of soft rocks has been largely discussed in the literature (Terzaghi & Peck, 1967; Rocha, 1975; Dobereiner, 1984; Johnston, 1993; Gonzales de Vallejo et al., 2002; Kanji, 2014). The lower limit of uniaxial compressive strength for soft or very soft rocks has been advocated by several authors as 2 MPa and the upper, around 20 MPa.

In Brazil, some dams have been built on sedimentary rock masses, which usually present greater permeability and deformability in comparison to metamorphic and igneous rocks – other examples of materials that also constitute the foundation of many dams around the world. Foundations formed by soft rocks – such as sedimentary masses – can even be suitable for gravity dam's construction because a dam failure is the product of more factors than just the foundation geology type. Although, in such cases, a large deformation of the foundation can take place during the dam construction and its operation period.

It is essential to evaluate the dam massif safety and its foundation through the prediction of pore pressures and deformations levels during the design stage. In addition, it is also necessary to investigate the dam massif performance during its construction and operation period, to verify the safety at field and confirm that the assumptions adopted in the design stage were appropriate. The investigation process related to dam performance is commonly done through the auscultation by in situ hydro-geotechnical instrumentation, which includes the measurement of pore pressures, flows, deformations, and displacements.

[#]Corresponding author. E-mail address: verleioliveira@gmail.com
[1]Universidade de Brasília, Brasília, DF, Brasil.
[2]Universidade Federal do Rio Grande do Sul, Porto Alegre, RS, Brasil.
[3]Companhia Estadual de Energia Elétrica do Rio Grande do Sul, Departamento de Geração e Transmissão, Porto Alegre, RS, Brasil.

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The comparison with the performance of other similar dams is important for safety analysis based on dam's behavior experience. But in the case of dams built on sedimentary rocks, only a few instrumentation data from other structures can be found in the literature to help professionals involved in dam monitoring. Within this context, this paper describes a case study whose goal is to present and analyze the main data related to the monitoring of foundation behavior of the Dona Francisca hydroelectric power plant (HPP) gravity dam, whose foundation is essentially constituted by sedimentary rocks. The foundation behavior analysis was done in terms of pore pressures and deformations recorded during seventeen years of dam operation by vibrating wire piezometers and multiple rod extensometers, respectively.

The rock formation at the Dona Francisca dam site is very stratified and made up mainly of sandstones, with also intercalated levels of siltstone and argillite. Due to the concern about the behavior of this type of foundation in relation to the stability of the structures, the Dona Francisca HPP was contemplated with a broad instrumentation plan, composed mainly of direct pendulums, multiple rod extensometers, triorthogonal joint meters, foundation piezometers, v-notch weirs, vibrating wire piezometers and thermometers. In this paper, only the vibrating wire piezometers and multiple rod extensometers data were analyzed because they are instruments that allow directly monitoring the dam foundation behavior over time.

2. Dona Francisca dam

The Dona Francisca dam was built at the end of 2000, on the Jacuí River, in the central region of the Rio Grande do Sul State, Southern Brazil. Its structure is constituted by rolled compacted concrete (*RCC*) and has approximately 63 meters of maximum height and 660 meters in length. The Dona Francisca dam (Figure 1) is part of the Jacuí power generation system and has an installed capacity of 125 Megawatts from two Francis turbines. The construction was started in August/1998 and the reservoir was filled in November/2000, when in one week it reached a quota of 94.50 m, corresponding to the crest of spillway.

2.1 Geology and geomechanical parameters of the foundation

The geological and geotechnical conditions of the dam foundation in the spillway region of the Dona Francisca HPP are associated to the rocks from Formação Caturrita, belonging to the Rosário do Sul Group, of the Paraná Basin. However, the dam abutments are in contact with basaltic rocks so that the power station building, and water intake blocks are set on these rocks. In terms of geological-geotechnical conditions, two of the initial concerns during the design phase were the geomechanical parameters of the Formação Caturrita rocks and the contact regions between these and the basaltic massifs.

Antunes Sobrinho et al. (1999) describes the dam foundation material as predominantly composed of fine-grained to medium-sized arkose sandstones, reddish to grayish color, massive or cross-stratified, found at varying degrees of lithification due to variations in nature and in the amount of the cementitious material. Interspersed to the sandstones there are also sub-horizontal layers of siltstones, argillite and resistant and persistent intraformational breccia / conglomerates, with thicknesses ranging from centimeters to meters and lateral extension of meters to tens of meters. In some locals, siltstones and weathered argillites were sampled showing characteristics of soils at different levels, ranging from millimeters to tens of centimeters – generally from 1 to 20 cm.

The foundations near the dam base in the riverbed region are composed by sandstones of the Formação Caturrita with degrees of coherence A-I and A-III. As an example, the Figure 2 shows the geological section of block B21 located in the spillway region with a stratified profile in this location. There were found sandstones with unconfined compressive strength (*UCS*) greater than 10 MPa for A-I condition and

Figure 1. Panoramic view of the Dona Francisca dam.

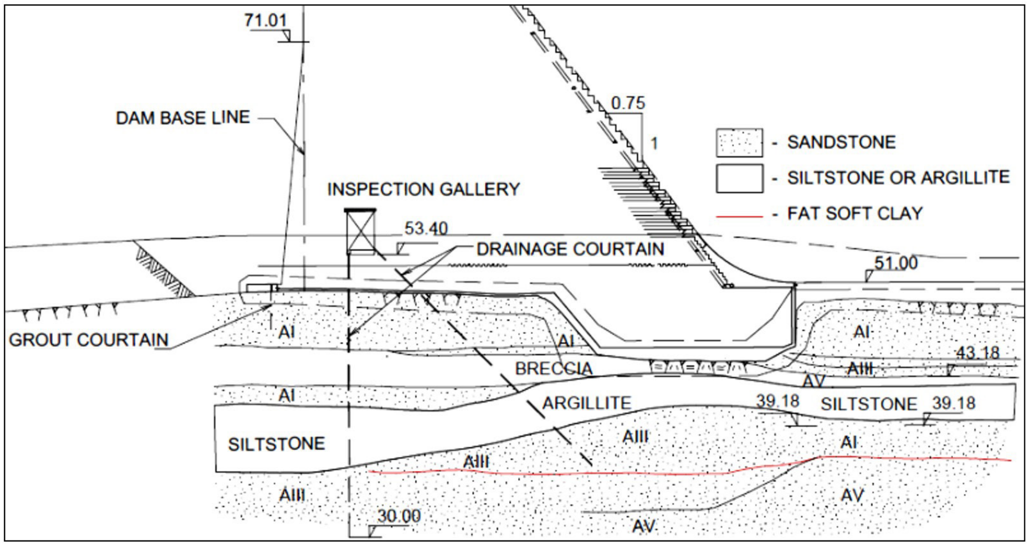

Figure 2. Geological section of block B21 foundation in the spillway region.

Table 1. Results of the uniaxial compression tests carried out on siltstone and argillite samples (adapted from Antunes Sobrinho et al., 1999).

Type of Rock	Sample number	Specific weight (g / cm^3)	*UCS* (MPa)	Modulus (MPa)	
				Secant	Tangent
Siltstone	5	2.233	9.1	707.0	941.0
	6	2.324	5.4	984.0	1200.0
	7	2.312	7.2	733.0	991.0
	11	2.372	9.5	937.0	990.0
	12	2.386	7.1	1157.0	902.0
	Mean	2.325	7.7	903.6	1004.8
Argillite	8	2.361	4.0	620.0	770.0
	14	2.364	8.3	994.0	750.0
	9	2.155	4.6	725.0	766.0
	10	2.270	3.8	390.0	344.0
	Mean	2.288	5.2	682.2	657.5

varying from 2 to 10 MPa in A-III condition. Laboratory tests indicated that the siltstone *UCS* ranged from 7 to 20 MPa and from 4 to 8 MPa for the argillite.

According to Pastore et al. (2005), the adoption of an *RCC* dam implies in rigorous foundation requirements. The authors also mention a particularly critical situation in the sliding stability verification and in the dam profile definition. It all depended on the determination of the behavior of the foundation rocky package and its geomechanical characteristics – mainly involving the layers of siltstone and argillite.

The Table 1 presents the results of the uniaxial unconfined compression tests carried out on siltstone and argillite samples during the design phase in 1998.

The graphs in Figure 3 show the strength envelopes, indicating the values of cohesion (*c*) and friction angle (φ) for these materials.

From the results presented in Table 1 and in the graphs of strength envelopes in Figure 3, it is observed a large variation in geomechanical parameters of these rocks. After several studies and tests carried out on the materials which compose the Dona Francisca dam foundation, the designers in charge had the task of defining the geomechanical parameters to use in stability calculations. This task was not easy for the design team because the foundation is composed by stratified sedimentary rocks and little was known about other dams built in these conditions. Thus, conservative criteria were adopted to define the geomechanical parameters during the stability analysis carried out by the engineers and consultants. The Dona Francisca HPP had a careful design of hydrogeotechnical instrumentation also due to the concern with the behavior of this type of foundation in terms of dam's massif stability.

2.2 Instrumentation "key sections"

The instrumentation of the Dona Francisca dam's massif is composed by direct pendulums, multiple rod extensometers, triorthogonal joint meters, foundation piezometers (standpipe and vibrating wire piezometers), v-notch weirs and thermometers. In terms of deformation monitoring – in addition to the triorthogonal joint meters installed along the dam – "key sections" were defined at blocks IW (intake works region), B17, B21, B25 and B28 for direct pendulums or extensometers installation. These blocks were chosen strategically according to location or because they had the highest heights. In blocks B17 and B21, direct pendulums were installed to monitor crest displacements. In blocks IW, B25 (in the spillway) and B28 (in the left abutment) multiple rod extensometers (*MRE*) rosettes were installed. In blocks IW, B06, B17, B21, B25, B28 and B30 vibrating wire piezometers (*VWP*) were installed to monitor the foundation pore pressures.

From the peculiar geological and geotechnical conditions already reported, this paper aims to describe the behavior of the Dona Francisca HPP dam foundation throughout time from the records of pore pressures and deformations, obtained from instruments installed at the abutments and the spillway. The Figure 4 presents a general arrangement of the dam concrete blocks divided into blocks with an average length of 20.0 m and indicating the "key sections" location where the *MRE* were installed.

The Figure 5 shows the rosettes of extensometers set on the blocks IW and B25 of the Dona Francisca dam. The block B28 (left abutment) has the multiple rod extensometers arranged similarly to those in block B25. It is also observed in the region of intake works one extensometer (MRE-IW-04) that monitors foundation deformation right below of two penstocks.

The Figure 6 indicates the "key sections" location where the vibrating wire piezometers were installed.

The Figure 7 shows, as an example, the relative positions of the vibrating wire piezometers installed in block B30. One of the *VWP* is willing upstream at the dam base and two are willing downstream – one at the dam base and the other at the foundation rock mass. Others blocks section has the *VWP* arranged similarly to those in block B30.

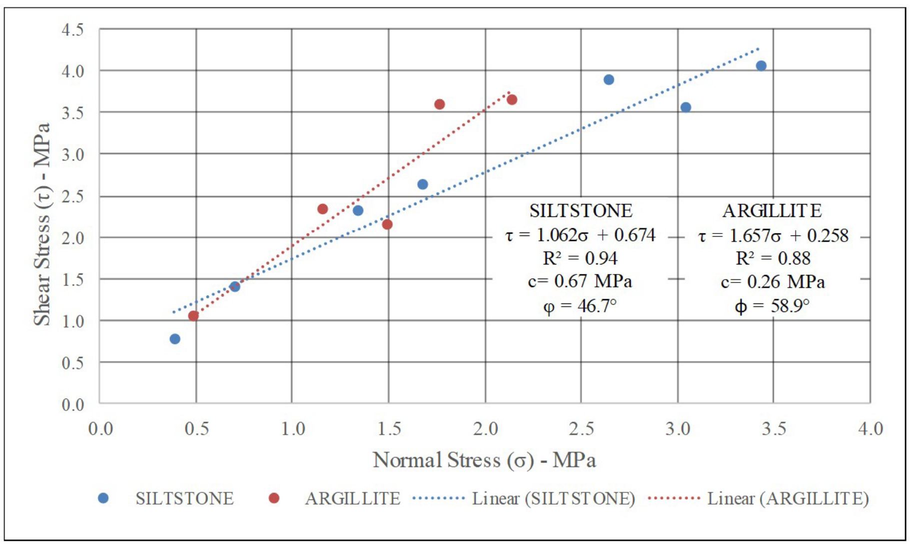

Figure 3. Resistance envelopes line of siltstone and argillite (adapted from Antunes Sobrinho et al., 1999).

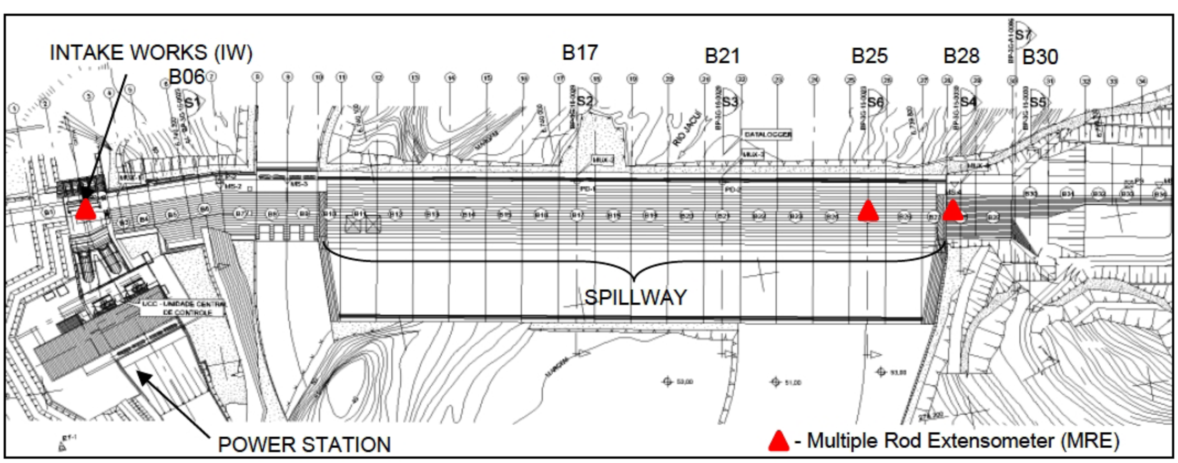

Figure 4. General arrangement of concrete blocks of the Dona Francisca HPP and MRE location.

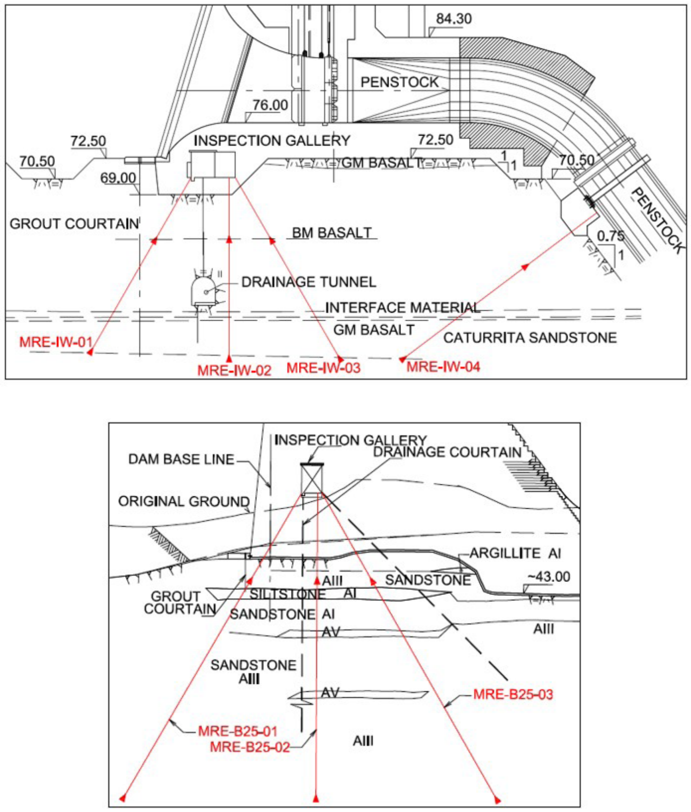

Figure 5. Multiple rod extensometers placed in the IW and B25 blocks.

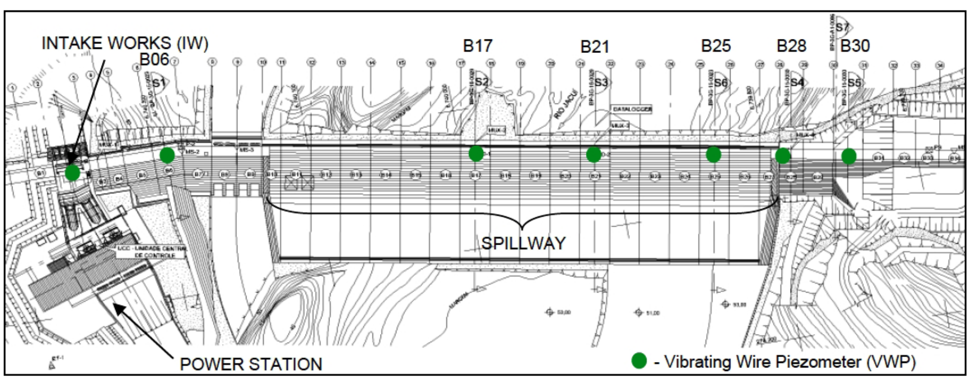

Figure 6. Key sections of Dona Francisca dam location where the VWP were installed.

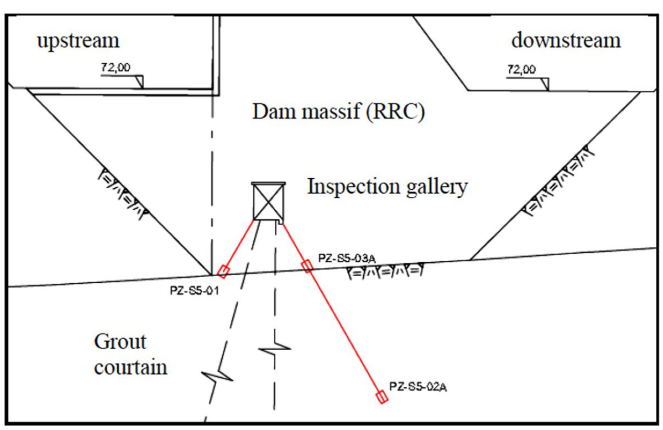

Figure 7. Vibrating wire piezometer's arrangement in block section B30.

3. Pore pressures analysis of Dona Francisca dam foundation

Data from the 18 vibrating wire piezometers (*VWP*) will be analyzed in this article. In the right abutment region, there are 2 *VWP* in the intake works section and 2 *VWP* in the S1 section, related to B06 block. In the spillway region there are 3 *VWP* in the S2 section, related to B17 block; 3 *VWP* in the S3 section, in B21 block; and 3 *VWP* in the S6 section; in B25 block. On the left abutment, there are 2 *VWP* in the S4 section, related to B28 block, and 3 *VWP* in the S5 section of B30 block.

The analysis in terms of hydraulic pressures in the foundation and at the base of the dam due to uplift water pressure (*U*) is carried out by converting the piezometric quota (*PQ*) into their corresponding loads, through the difference between PE and the installation quota (*IQ*) of each piezometer (Equation 1). The unit of measurement obtained for U is given in meters of water column (m H_2O).

$$U = PQ - IQ \tag{1}$$

Table 2 presents a comparison between the observed uplift pressures in each of the piezometers (*PZ*) and their design predicted values. The *PZ* highlighted in bold in the "piezometer code/block" column (second column) corresponds to the instruments installed at the dam base, facing downstream. The *PZ* highlighted in italics correspond to the instruments installed at the dam base, facing upstream. The other piezometers are located inside the rock mass.

According to design forecasts, the uplift pressures at the dam base facing downstream should be lower than the values recorded inside the foundation mass. About the uplift pressures at the dam base recorder by *VWP* facing upstream, they were estimated during design period disregarding the possible influence of the injection curtains in the overall reduction of uplift water pressures. In this way, their values are higher than the uplift water pressures recorded by downstream *VWP*. This may also justify the control parameters and pore pressure limits being high in relation to what has been observed in these 17 years of monitoring.

However, the influence of injection curtains in reducing uplift water pressures is still a controversial topic. It is known that they are essential to reduce seepage through the foundation and, consequently, can increase the effectiveness of drains – especially in a very permeable rock mass – since otherwise the drains could be overloaded with excessive flows. On the other hand, Costa (2012) comments that in areas of low permeability of a dam foundation, injections do not have any "positive effect", regardless of the purpose for which they were programmed. Injections should reduce percolation in very permeable areas, but only drainage will relieve uplift water pressure. Therefore, the best solution tends to be the simultaneous use of these two systems – injection curtains and drains.

The graphs in Figure 8 and Figure 9 show the variation in uplift water pressures over the 17 years of monitoring the *VWP* in the inspection gallery region and in the stilling basin, respectively.

From the graph in Figure 8, it is highlighted that the highest pressures currently tend to be registered in VWP

Table 2. Hydrostatic uplift pressures comparison (design *versus* observed loads).

Dam region	Piezometer code / block	*U* (m H_2O)		Umax: greatest record (mH_2O)	Current trend of *U* (mH_2O)
		Control	Limit		
IW/RA	PZ-TA-01 (downstream)	1.43	11.43	1.13	0.63 to 0.93
	PZ-TA-02 (downstream)	0.60	0.60	0.80	-0.40 to 0.70
RA	PZ-S1-01 (downstream) / B06	19.49	32.39	6.69	3.29 to 4.09
	PZ-S1-02 (downstream) / B06	11.71	23.91	4.61	0.81 to 1.81
Spillway	PZ-S2-01 (downstream) / B17	21.50	23.50	4.20	1.60 to 2.10
	PE-S2-01 (stilling basin) / B17	-	-	8.95	1.45 to 2.45
	PE-S2-02 (stilling basin) / B17	-	-	12.85	2.25 to 3.75
	PZ-S3-01 (downstream) / B21	16.50	17.50	3.40	2.80 to 3.10
	PZ-S3-02 (downstream) / B21	9.04	9.98	3.84	3.34 to 3.54
	PE-S3-01 (stilling basin) / B21	-	-	9.60	2.00 to 4.00
	PZ-S6-01 (upstream) / B25	24.21	28.21	11.61	6.61 to 7.61
	PZ-S6-02 (downstream) / B25	11.80	12.50	4.30	3.80 to 4.20
	PZ-S6-03 (downstream) / B25	8.64	9.64	4.14	3.54 to 3.94
LA	PZ-S4-01 (downstream) / B28	23.64	37.04	7.54	7.14 to 7.34
	PZ-S4-02 (downstream) / B28	11.31	23.61	4.71	3.11 to 4.11
	PZ-S5-01 (upstream) / B30	31.60	38.80	8.70	5.90 to 6.30
	PZ-S5-02 (downstream) / B30	15.61	28.21	1.41	0.51 to 0.91
	PZ-S5-03 (downstream) / B30	2.5	16.1	9.90	9.10 to 9.40

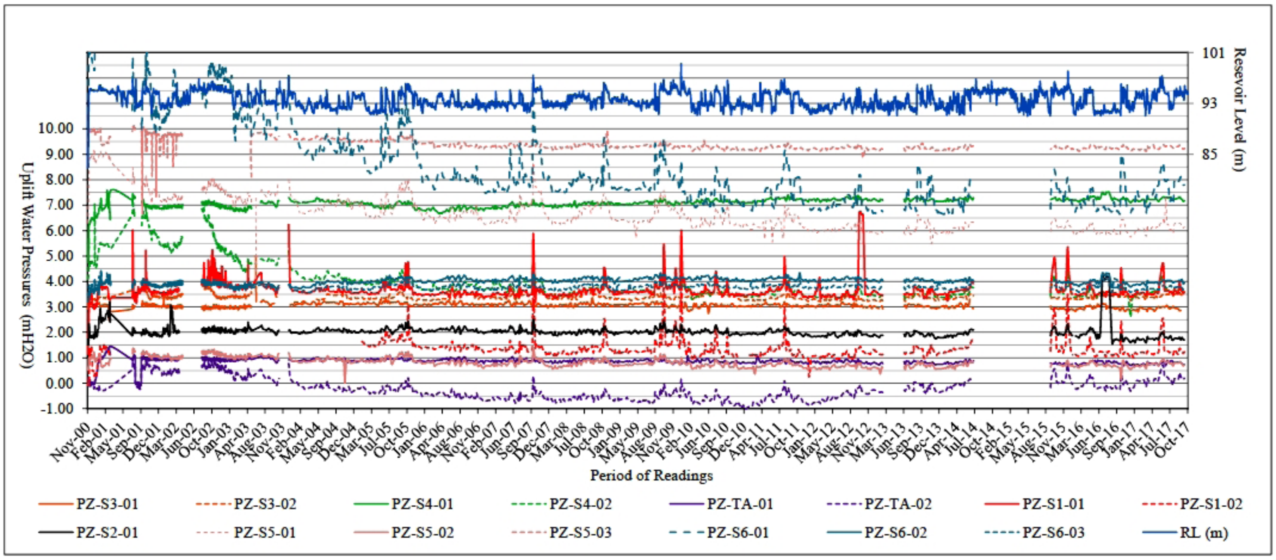

Figure 8. Variation of uplift pressures in the inspection gallery *VWP*.

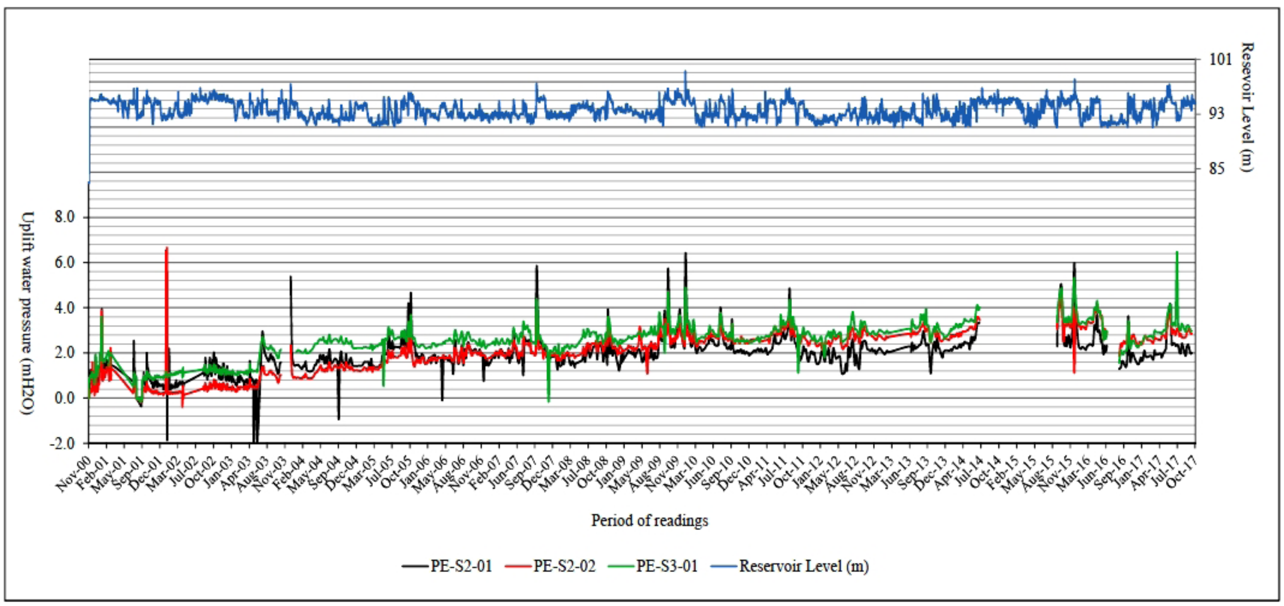

Figure 9. Variation of uplift pressures in the stilling basin *VWP*.

PZ-S5-03 and PZ-S6-03, both facing upstream, and it is also observed that under normal operating conditions there is an uplift pressures oscillation trend as a reservoir level upstream (RL) function. The uplift pressures observed in the stilling basin piezometers (Figure 9) present the same order of magnitude and increased until mid-2009. From then on, their records stabilized between 1.5 and 3.0 m H_2O.

It is interesting to observe how quickly the system responds. In July 2016, during a maintenance procedure to carry out an inspection of the stilling basin, the pumping system of well N. 02 – located on the spillway – was shut down for a few days. The flows that discharged in well N.02 were diverted to well N. 01. As a result, there was an increase in the water level inside well No. 2 and the uplift pressures in this region increased immediately after the pumps were turned off. On this occasion, several piezometers reached their historic maximums.

The uplift pressures verified in the right abutment region (*RA*) from the records at the dam base (contact of the *RCC* with the basaltic or sandstone rock) are in the same instrumented section lower than the uplift pressures inside the rock mass underlying the dam base.

The uplift pressures in the *RA* piezometers tend to be lower than those of the control or slightly higher (~ 0.10 m.c.a.), as in the PZ-TA-02 case. Its lower record of the current reading trend is a slightly negative uplift pressure (- 0.40 m.c.a.), which reveals that the region is well drained – close to the drainage tunnel. In general, the conditions referring to the piezometry data in this region can be considered normal.

In the region of the stilling basin, the PE-S2-01, PE-S2-02 and PE-S3-01 *VWP* present uplift pressures in the same order of magnitude. Considering the trend of stability of these *VWP* readings, it is possible to infer that the uplift pressure conditions in the stilling basin region were always within the expected behavior, except when the well drainage pump No. 2 was turned off. Unfortunately, the control or limits values for these *VWP* were not estimated in design phase so that some comparison could be made with the database.

The uplift pressures observed in the spillway region tend to be lower than the control values. The pore pressures at the downstream dam base – indicated by PZ-S3-02 and PZ-S6-03 *VWP* – are of the same order of magnitude and appear to be stabilized. In section S3, the uplift pressure at the dam base is slightly higher than that recorded inside the foundation mass by PZ-S3-01 *VWP*.

In section S6, the uplift pressure at the downstream dam base (PZ-S6-03) is of the same order of magnitude as that observed in PZ-S6-02 instrument, which is installed inside the sandstone mass. Since the uplift pressure at the upstream dam base – indicated by PZ-S6-01 – is approximately twice the one observed at the downstream dam base by PZ-S6-03, it is possible to infer that the drains in this region are reducing the pressures by about 50%. It should be noted that the injection curtain was not considered in the design phase when estimating the control and limit piezometric quotas and, given the possibility of its influence, it is justified that the control and limit parameters are relatively high. In general, the conditions from the piezometry in this region can be considered normal.

4. Deformation analysis of Dona Francisca dam foundation

The data of foundation deformation of a concrete dam consists in one of the most important information in the monitoring of the behavior of these structures, throughout the time. These deformations result from the loads imposed by the construction of dam's massif, reservoir filling and variations in reservoir level during operational period. The measured settlements in the construction period and in the first years of operation of the dam allow a first assessment of the real deformability of the foundation, through comparison with design estimates.

Extensometers are the most indicated instruments for direct monitoring of deformation that occur between the anchor points and the head of the rods of these instruments installed in the foundation. Typically, they are installed from the inside of drainage gallery, close to the base of the dam. In the case of Dona Francisca dam, the settlements during constructive period were not measured. However, after this stage, multiple rod extensometers (*MREs*) were installed in key sections of blocks IW, B25 and B28, allowing the monitoring of deformations due to the filling of the reservoir and during operational phases.

The settlement, resulting deformation and horizontal translation at the base of a concrete dam can be estimated from the installation of a rosette of extensometers in a given block. The rosette is a set of three RMEs installed from the inspection gallery, in which the vertical is at the center and the other two are inclined approximately 30 ° in relation to the vertical. The *MREs* installed were made of fiberglass rods and mounted on site. Each rod was inserted into a rotary drilling hole, fixed at its lower end through a grout-injected section, and the upper end was placed in an easily accessible location – in the region of the inspection gallery. A polyethylene tube protects each rod by avoiding contact with the grouting. The standard diameter of the rods is 8.0 mm and each MRE has two rods: a short rod (*SR*) with a length of 7.0 to 10.0 m and a long rod (*LR*) with a length of 30.0 to 40.0 m. The function of the SR is monitoring the deformation of the rocky massif right below the base of the dam.

The Figure 10 present the evolution of deformations recorded by the long rods of the *MREs* installed in B25 and B28 blocks, and the Figure 11 present the evolution of deformations recorded by the long rods of the *MREs* installed in *IW* block – during about 17 years of monitoring. The negative values of deformations correspond to distension of the rods as a consequence of stresses relieving in the rocky massif at foundation. The positive values of deformations are related to compression of the rods, as consequence of the compression in the rocky massif at foundation. Some gaps observed in deformation records occurred due to technical problems with the *MREs* reading equipment, making it impossible to monitor during this associate period.

Due to the horizontal loading, resulting from elevation of the reservoir level, it is expected that stress relief will occur and hence uplift/elevation of the rocky massif at the upstream dam base, in addition to the increase of stresses and settlements at the downstream dam base. This can be verified from the correlation shown in the Figure 10 and Figure 11, linking deformations and the oscillation of the reservoir level all over time. It is observed that the *MREs* placed at upstream zone of the dam are more sensitive to the *RL* oscillation.

Through the analysis of deformation data of the foundation, acquired from the extensometers positioned in the IW block and shown in the graphs of Figure 11, it is possible to verify that the magnitudes of the records are lower to those found in the blocks of the spillway region (B25) and left abutment (B28). The reason for this behavior is that the foundation at the IW block region is composed of basaltic rocks which present lower deformability in comparison to the sedimentary rocks which are predominant in other

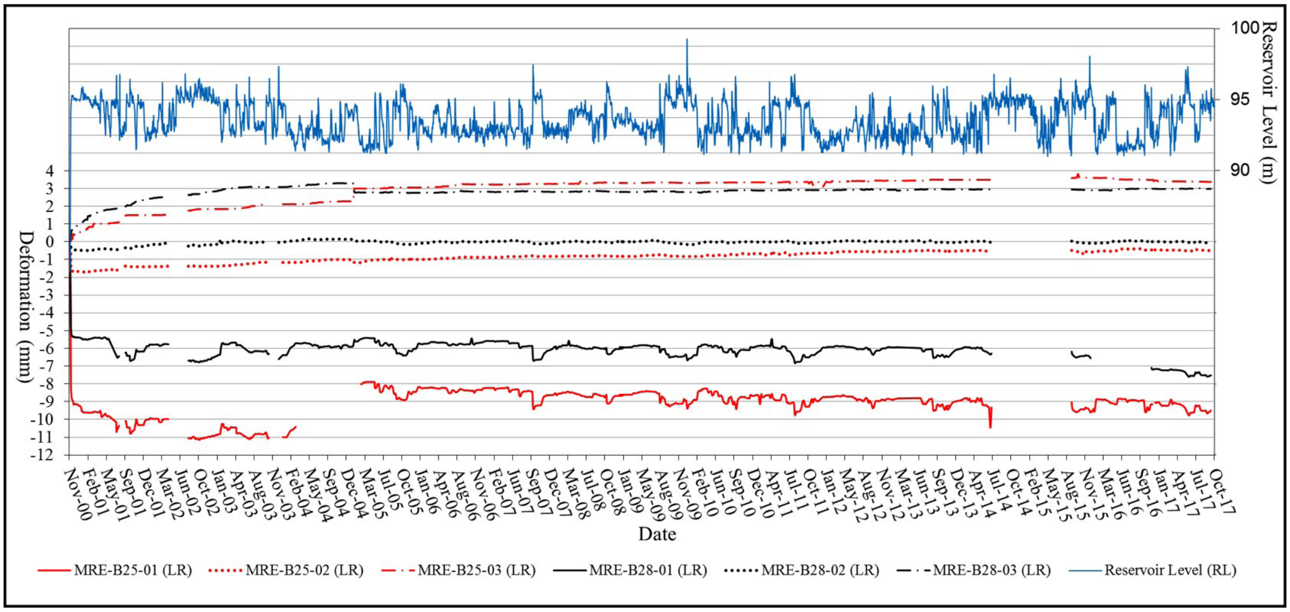

Figure 10. Total deformation recorded from the *MREs* long rods placed in the B25 and B28 blocks.

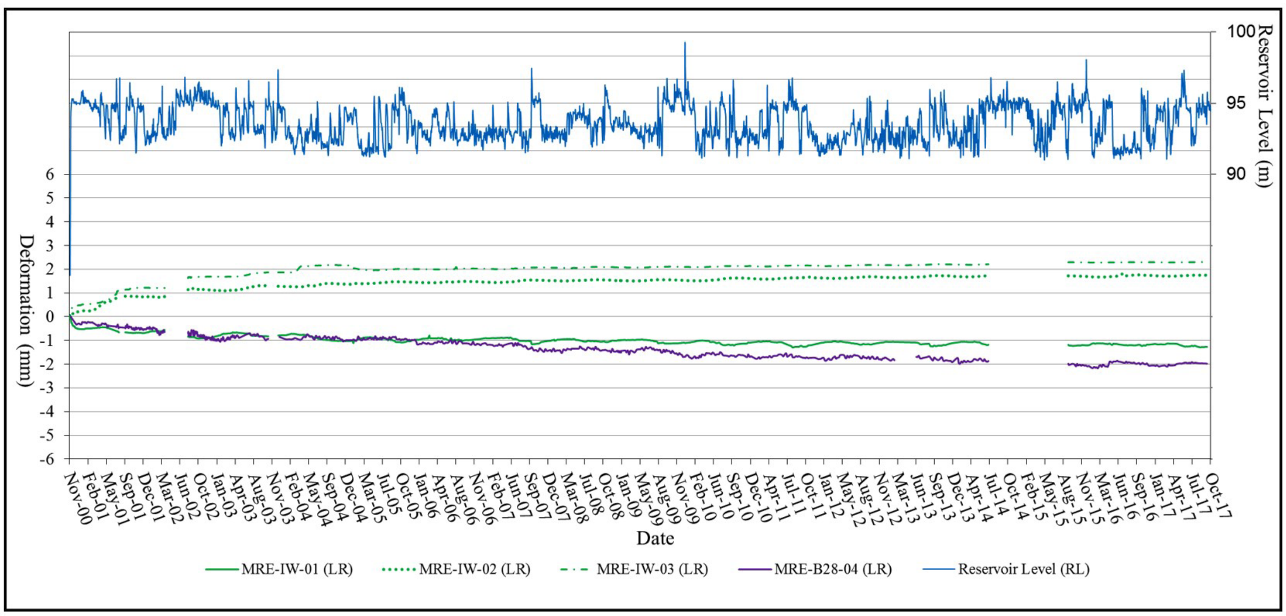

Figure 11. Total deformation recorded from the MREs long rods placed in the IW block.

regions of the dam. In general, the deformation data of the Dona Francisca dam foundation are consistent in terms of expected behavior.

The upstream directed extensometers (*MREs*-01) have always been stretched. The extensometers installed vertically (*MREs*-02) tended to indicate either small compression, or distension. The *MREs* at downstream (*MREs*-03) have always been stretched. However, it should be noticed that the theoretical design predictions for the extensometer readings indicated distension values even for the instruments placed at downstream sections of the dam, highlighting the difficulty of predicting the behavior for this specific type of foundation. In terms of magnitude of deformations, most of the design estimates were lower than the actual structure behavior.

The Table 3 presents a summary of the elastic (δe) and total (δt) deformations observed in the *MREs* installed in the Dona Francisca HPP dam. The data refer to the initial period

Table 3. Summary of the elastic (δe), slow (δs) and total (δt) deformations recorded from the *MREs*.

MRE / LR – Long Rod / SR – Short Rod		Δe (two weeks)		δt = δs + δe				
				5 years (Jan - 2005)			17 years (Out - 2017)	
		δe* (mm)	δe/δt17 (%)	δt5* (mm)	δt5/δt17 (%)	Rate of δs - 2000 to 2005 (mm/year)	δt17* (mm)	Rate of δs - 2005 to 2017 (mm/year)
MRE-IW-01	LR	-0.28	22	-1.03	81	-0.150	-1.27	-0.020
	SR	-0.28	70	-0.25	62	0.006	-0.40	-0.013
MRE-IW-02	LR	0.18	10	1.36	78	0.236	1.75	0.033
	SR	0.14	25	0.58	105	0.088	0.55	-0.002
MRE-IW-03	LR	0.37	16	2.16	94	0.358	2.30	0.012
	SR	0.24	27	0.84	93	0.120	0.90	0.005
MRE-IW-04	LR	-0.27	14	-0.99	50	-0.144	-1.98	-0.083
	SR	-0.36	26	-0.94	69	-0.116	-1.37	-0.036
MRE-B25-01	LR	-8.86	93	-8.00	84	0.172	-9.50	-0.125
	SR	-4.84	46	-9.27	89	-0.886	-10.41	-0.095
MRE-B25-02	LR	-1.65	337	-1.17	239	0.096	-0.49	0.057
	SR	-0.71	473	-0.32	213	0.078	-0.15	0.014
MRE-B25-03	LR	0.15	4	3.00	89	0.570	3.37	0.031
	SR	0.85	23	3.00	82	0.430	3.67	0.056
MRE-B28-01	LR	-5.30	70	-5.91	79	-0.122	-7.52	-0.134
	SR	-1.36	52	-2.04	79	-0.136	-2.59	-0.046
MRE-B28-02	LR	-0.40	667	0.14	-	0.108	-0.06	-0.017
	SR	-0.29	45	-0.43	66	-0.028	-0.65	-0.018
MRE-B28-03	LR	0.65	22	3.30	110	0.530	2.99	-0.026
	SR	0.12	27	0.52	118	0.080	0.44	-0.007

* Compression (+) or distension (-).

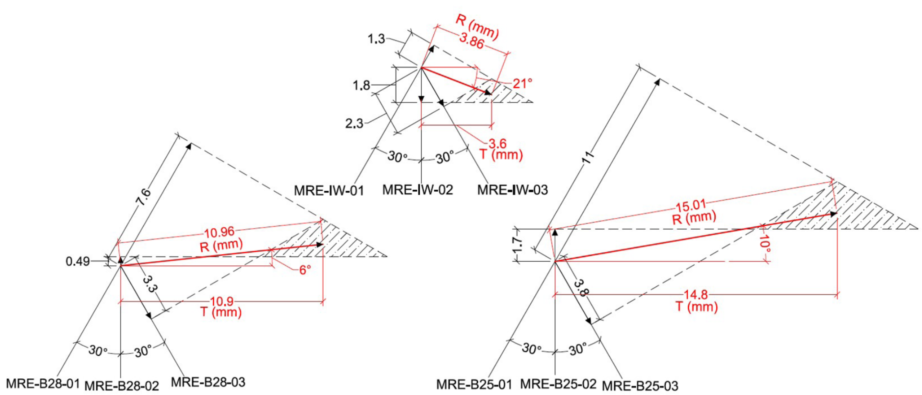

Figure 12. Resulting maximum deformation (*R*) and horizontal displacement (*T*) in the dam base.

and after 5 and 17 years of operation. The percentages – in relation to the total period of 17 years of monitoring – of δe and δt after 5 years of operation are presented. In addition, the rate in which the slow deformations (δs) occurred in the first 5 years and past 12 years are showed below.

Analyzing the previous graphs from Figure 10 and Figure 11 and the data presented in Table 3, it is observed that the process of settlement of the rocky massif was more intense in the first 5 years of dam operation. From that period on, although the deformations continued to occur, mainly in the deeper layers of the foundation, it was observed that it took place in a lower rate. At the end of 17 years of monitoring, it is possible to notice the tendency of a final stage of stabilization of the rocky massif. The experience reported by some authors – mainly regarding dams on basaltic massifs – shows that these long-term or slow deformations can occur in periods superior to 20 years after reservoir filling – such as the cases of Itaipu and São Simão – Brazilian dams presented in Silveira (2003).

The Figure 12 presents the vector calculations of the resulting maximum deformation (R) and horizontal displacement (T) of the Dona Francisca HPP dam base, from the maximum deformations recorded by the MREs rosettes installed in blocks IW, B28 and B25.

According to Silveira (2003), since the lower anchoring points of the extensometers are in an absolute reference, calculating a composition of the distension or compression recorded by the three *MRE*s (long rods) results in a triangle and not a point. For the calculation of the resulting deformation, this same author recommends the determination of the geometric center of a triangle formed by the intersection of the three bisectors of the projected vectors. From the vector calculations presented, it is possible to estimate that the resultant deformations (R) and the horizontal displacement (T) at the dam base in the spillway region were of the order of 10.0 to 15.0 mm – higher than that one verified for the intake works region, whose value was approximately 4.0 mm.

5. Conclusions

Argillites, siltites and sandstones are sedimentary rocks that are commonly referred to as soft rocks because of their low resistance. According to Kanji (2014), these rocks are a critical material since they present a series of problems. They have a behavior intermediate between soil and hard rock and often, they cannot be tested neither in soil mechanics laboratories due to its high resistance, nor in rock mechanics laboratories as they are too soft to be trimmed and tested. Because of soft rocks' complexity in structure, material property, and geological environment, knowledge and understanding about soft rocks is a challenge in geotechnical engineering, mainly in the case of dams built on these massifs.

The comparison with the performance of other similar dams is important for safety analysis based on dam's behavior experience. But in the case of dams built on sedimentary rocks, only a few instrumentation data from other structures can be found in the literature to help professionals involved in dam monitoring. Within this context, this paper brings a case study about the geotechnical behavior of Dona Francisca gravity dam that was built on sedimentary rocks.

The deformability of rocky masses – related to a certain loading – is influenced by four factors: mineralogical composition, alteration degree, discontinuity plans and the relationship between the direction of the load applied and the discontinuity direction. These influences could be verified from the deformations observed in the foundation of Dona Francisca dam, since they presented a higher magnitude than most of the records of other Brazilian dams, reported in Silveira (2003).

This fact is mainly related to the foundation characteristics of Dona Francisca dam that usually present larger deformability than, for example, the basaltic masses on which several dams were built in Brazil. Therefore, the construction of dams in such foundation conditions requires special attention from the design stage, especially through the implementation of a good instrumentation plan, allowing the monitoring of structure behavior in a continuous way and evaluating its safety.

The deformation data recorded from the extensometers are consistent and enable the describing of foundation behavior of Dona Francisca HPP dam. The influence of time factor can be observed on the deformations of the rock mass when permanent loading due to self-weight of the dam and hydraulic loadings are associated. The rock layer near the dam base and monitored by the short rods of the extensometers was practically stabilized after the first 5 years of operation. The deeper layers of foundation, monitored by long rods, tended to be in a final stage of stabilization, indicating a normal situation – although a small rate of deformation (0.025 mm/year) can be observed.

The uplift pressures at the base and foundation of the Dona Francisca HPP dam tended to decrease and / or stabilize over time. In this way – except for specific situations that presented some unexpected situation – the scenario obtained in terms of structure safety from the piezometric analysis can be considered normal.

Finally, it is worth emphasizing the importance of carrying out the dam's hydrogeotechnical instrumentation analysis together with the observations obtained from the visual inspections, which are also relevant to guarantee adequate control and comprehensive monitoring of the concrete dam behavior.

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Declaration of interest

The authors have no conflicts of interest to declare.

Authors' contributions

Verlei Oliveira dos Santos: Conceptualization, Data curation, Investigation, Formal analysis, Methodology, Validation, Writing – original draft. Luiz Antonio Bressani: Conceptualization, Methodology, Supervision, Project administration, Validation, Visualization, Writing – review & editing. Camila de Souza Dahm Smirdele: Project administration, Data curation, Resources, Supervision, Visualization.

Data availability

Data generated and analyzed in the course of the current study are not publicly available because they are owned by

CEEE-GT. However, a complete or limited dataset can be made available upon reasonable request.

List of symbols

c	cohesion
HPP	Hydroelectric power plant
IQ	Installation quota
IW	Intake works
LR	Long rods
MRE	Multiple rod extensometers
PQ	Piezometric quota
PZ	Piezometer
R	Resulting maximum deformation
RA	Right abutment region
RCC	Rolled compacted concrete
RL	Reservoir level upstream
T	Horizontal displacement
U	Uplift water pressure
UCS	Unconfined compressive strength
VWP	Vibrating wire piezometer
δe	Elastic deformation
δs	Slow deformation
δt	Total deformation
σ	Normal stress
τ	Shear stress
φ	Friction angle

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