STUDY AND MONITORING THE CONSTRUCTION OF A CONCRETE FACE ROCKFILL DAM (CFRD)

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Abstract*. Concrete face rockfill dams (CFRDs) are becoming a widely used type of rockfill dam all over the world. However the design and construction of CFRDs are based primarily on precedent and engineering judgments. Few numerical analysis methods have been developed to properly evaluate the deformation of CFRDs.*

This paper describes the setup of finite element method (FEM) models for the three dimensional and two dimensional simulations of the construction of a concrete face rockfill dam and first filling of its reservoir using the Code-Aster code. This work also includes a seepage analysis to predict the flow through the foundation and rockfill embankment. The prototype of the study is the 36.5 m high Montesinho dam, located in the north of Portugal near the Spain border. This dam is finishing its construction, at the time of the paper writing. In this study, a finite element procedure was developed to simulate the construction process of the dam, the first filling and seepage analysis. An elastic perfectly plasticity model was used to model the rockfill materials. The model parameters were calibrated by large-scale triaxial tests performed on materials used in the dam. The step-by-step construction followed by subsequent impounding of the reservoir was simulated in the numerical procedure. The numerical results agree well with in situ monitoring records of dam settlements, indicating that the finite element procedure developed can be used to evaluate the deformations of CFRDs.

1 Introduction

Concrete face rockfill dams had its origin in the mining region of Serra Nevada in California in the 1850's (ICOLD, 2010). Since the beginning of the use of rockfill in dams, numerous breakthroughs have been achieved. The use of vibratory-rollers has revolutionized the way this type of dam was built and substantially increased its performance. With the increasing number of CFRD's, experience level and designers confidence in this structure has increased. Thus, in recent decades increasingly high dams have been constructed. However the design guidelines were maintained essentially empirical. Problems of excessive deformation and cracking of the reinforced concrete curtain were recorded in several very high dams. This recent problems reveal the need to combine the numerical calculations with the theoretical and laboratory studies and monitoring data in order to achieve sustainable development in this particular type of dams.

The main objective of this thesis is to develop a set of numerical models to simulate the behavior of CFRD in different phases of their life cycle. The models were developed through the use of "open source" software. In this sense it is intended, on the one hand, to reduce the dependence of license contracts, on the other to ensure the longevity of the developed models. It was also sought to build a fully automated and parameterized model enabling rapid application to different CFRD dams.

In the following chapters, first a short introduction to concrete face rockfill dams is given. Chapter 3 presents a quick description of Montesinho dam. The following chapter – 4 – briefly addresses the Salome-Meca bundle. In chapter 5 all the steps to setup the models of the dam are presented. In chpter 6 the results obtained are presented and discussed. Finally chpter 7 includes conclusions and suggested further developments.

2 Concrete Faced Rockfill Dams

Concrete Faced Rockfill Dams, CFRD, is a term used to describe a certain type of dams that is composed by a rockfill embankment with a waterproofing upstream reinforced concrete curtain. Besides the rockfill embankment and the concrete curtain CFRD's include a plinth, perimetral and vertical joints and have associated a foundation treatment. Figure 1 presents the construction of Montesinho CFRD, the case study of this paper, in two different stages.

Due to several advantages of CFRD's, including their adaptation to topography and geology, use of locally available materials, cost-effectiveness, simple construction and short construction period, they have been quickly developed in recent decades, with some reaching 200 m high (Xu et al., 2012)

Figure 1: Montesinho Dam: (a) rockfill embankment construction; (b) curtain construction with slip form.

CFRD are extremely stable structures, but their behavior can be conditioned by the deformability of the massive rockfill, since the deformations occurring therein can pass to the concrete curtain. Deformations of CFRD dams start occurring during the construction. These deformations are caused by the increase of effective stresses during the construction by the consecutive layers of rockfill material and also by creep effects. Deformations may also be influenced by deformations of the foundation, by transfer of stresses between the various zones of the dam and by other factors. After the construction is completed, considerable movements of the crest and of the body of the dam can develop due to the first filling of the reservoir. The load of water and the deformations of the rockfill of the dam force concrete curtain to deform. The concrete slab acts as an impervious membrane and any development of cracks in the slab would allow for water to penetrate the rockfill of the dam and could compromise the dam operation (Szostak-Chrzanowski et al., 2008).

3 Description of Montesinho dam

The Montesinho dam is located in the Sabor river in the Montesinho Natural Reserve. It is a concrete face rockfill dam (CFRD) built to provide water supply to Bragança town reinforcing the current reserve of the Serra Serrada dam located approximately 3 km west of Montesinho. Montesinho dam has 36.5 m of height and a crest with a length of about 310 m and 7 m of width. The total volume of the embankment is of about 174 000 m³ and consists of granitic rockfill obtained from the quarries located upstream of the dam in the reservoir area.

The reservoir has a capacity of 3.69 hm³ (net volume of 3.53 hm³) with a flooded area of 35.8 ha and a catchment area of 10.1 km². The normal water level is at 1217.50 m and the maximum water level is at 1219.73 m. The freeboard is 1.37 m, therefore the crest is located at elevation 1221.10 m. The upstream and downstream slopes were originally inclined at of 1:1.5 (v:h). During the construction, it was decided to include a berm in the downstream slope.

In the area of the dam and reservoir the outcropping blocks and top layers of the bedrock consists essentially of a two-mica granite with coarse grain. The rock presents a generalized and mild to medium kaolinization of the feldspars. Therefore its mechanical characteristics correspond to a weathered granite (W3 and W4), with low mechanical strength and a low deformability modulus. At greater depths its quality increases (W2 to W3).

Figure 2 and Figure 3 present the plan and the cross section of the dam. As it can be seen from the figures, the valley is asymmetrical with a average inclination of 1:6,5 (v:h) above elevation 1200 m on the right bank and 1:2,6 (v:h) on both left and right banks below that elevation.

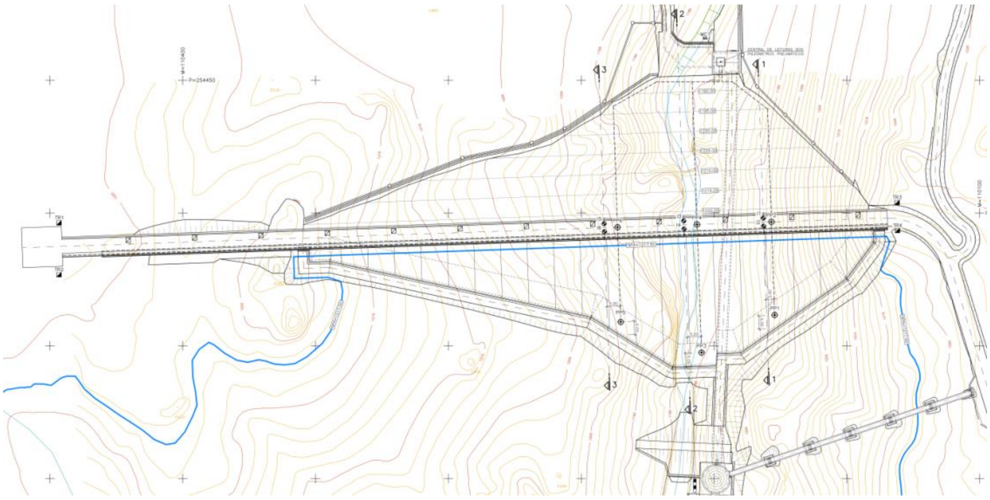

Figure 2: Plan of Montesinho Dam

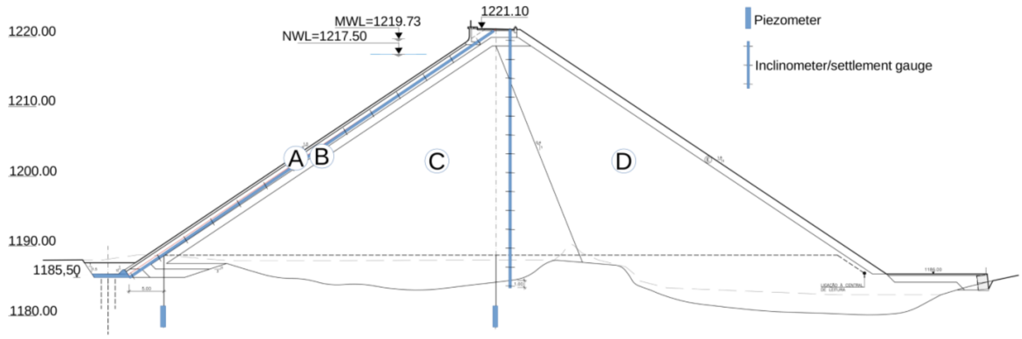

Figure 3: Montesinho dam cross-section with the layout of the settlement gauges

4 Brief description of Code_Aster and Salome-Meca software

Salome-Meca is a software bundle merging together several tools dedicated to the finite element method. It merges among others, the geometry module (Geom), the mesh module (Mesh), the command file editor (Eficas), the job control module (Aster) and the post-processing module (Paravis). The latter module is a fork of the well-known Paraview visualization software, with special import filters to process the native output from code-aster computing module. Salome-Meca runs natively in any modern Linux distribution. It can be installed directly using binary packages or compiled from source.

The Geom module consists of an object oriented CAD environment allowing to create arbitrary and complex geometries through the use of geometrical primitives, various transformation operations, boolean manipulation of objects, optimization algorithms, etc. The module is capable of importing several (open standard) file formats. As with all other modules in Salome-meca, most of the features of the Geom module can be addressed programmatically using python language.

The Mesh module reads the geometry and creates a finite element mesh using one of the algorithms and methods available. The generated meshes can include hexahedral, tetrahedral, triangular and quadrilateral elements for 3D and 2D geometries. Besides the mesh module allows for the creation of sub-meshes allowing refining the mesh density and creating groups of elements, faces or nodes to specify different materials and boundary conditions.

The Eficas module is dedicated to the creation of the script containing all the commands to the Code-aster modelling. This module is fairly complex and requires a good knowledge of the finite element method, because it permits to specify very complex analysis and operations to the model.

The Aster module is just a graphical user interface to launch the FEM calculations. It allows for the specification of the computing machine (local, remote or remote cluster), the specification of the number of cpus, memory allocation and total cpu time.

Finally, to view the results Salome-meca uses the Paravis module. Paravis derives from ParaView that is an extensible and very configurable framework used to view data in many forms.

5 Model setup

The finite element meshes and geometries necessary for its creation were developed using two scripts in the python programming language: one for 2D modeling and the other to a 3D model. These scripts are written so that the dam, the foundation and their meshes can be designed according to a series of inputs, introduced by the user. The modeling of the construction, the reservoir filling and the seepage analysis was created through Code_Aster command files that include all the calculation processes.

5.1 Geometry, scripts and automation

The geometry of the 3D model consists of 2 parts. The foundation is generated from a set of blocks with a triangular base and top. The foundation surface is obtained from a simple point sampling of the dam plan. Those points are submitted to a Delaunay triangulation creating the block set. All the blocks are finally fused together giving place to the foundation. All the process is easily obtained by means of a python script. The dam geometry is highly parameterized in the script. The user has to specify 2 points at the crest (the upstream edge) and all the relevant features of the dam geometry - crest and berm width, slopes, etc. The dam is then generated as a regular solid independent from the foundation. Using a boolean cut geometric operation this block is then cut by the foundation, adjusting its geometry to the real surface of the foundation. Because the stress and strain paths are relevant to the soils and rockfill behavior, the construction of the dam has to be made in layers (Naylor et al., 1981). The dam is then split in layers by means of several cutting planes, allowing for an arbitrary user-defined number of layers.

The 2D geometry was created using a similar process however the foundation was built as a regular face with the top at a constant elevation. Another difference between the two models geometry is that the 2D geometry takes into account the rockfill embankment and foundation zoning. The grout curtain was also defined in the 2D geometry in order to allow seepage calculations.

5.2 Finite element mesh

The mesh was generated using the simplest algorithm – Netgen (Schoberl 1997). In the case of Montesinho Dam analysis the generated mesh for the 3D model has about 111 k tetrahedral elements with 10 nodes. From those, about 35 k elements are used to build the 15 layers of the dam body. The 2D mesh is composed of approximately 3.3 k triangular elements of 6 nodes. In this module it is also advisable to define groups of nodes, faces and elements, to allow the imposition of boundary conditions and the construction sequence.

5.3 Modeling of the construction and the reservoir filling

The simulation of the construction of the dam is made in layers because it is necessary to account for the construction deformations and also because soil and rockfill materials behavior is dependent on the stress (and strain) paths. To account correctly to construction deformations one follows a Naylor et al. (1981) proposal, where the displacements of a newly constructed layer are dismissed. Using this method, when the dam is finally completed the displacements in the crest are null as in the reality, because the dam is always built up to the designed elevation. To model the filling of the reservoir two alternatives were considered. The first consists of modeling the water as special incompressible finite elements and the other on applying the water load. Considering all the implications of the former hypothesis namely a larger mesh, it was decided to consider the filling of the reservoir as loads applied in accordance with the reservoir level.

6 Material Characterization

The yield criterion used in the modeling of rockfill materials of Montesinho dam was the Drucker-Prager criterion. This model assumes a linear behavior in the elastic phase. However, it is considered that, for the low stresses to which the dam will be subject, the equivalent elastic modulus results obtained from the triaxial tests for a representative confinement stress are satisfactory.

The Drucker-Prager criterion uses in its formulation the I_1 and J_2 invariants. This formulation suggested by Drucker-Prager is considered a better approach for numerical modeling of the Mohr-Coulomb criterion and can be expressed by:

$$f = \sqrt{J_2} - \alpha \cdot I_1 - k$$

with:

$$I_i = \sigma_x + \sigma_y + \sigma_z$$

$$\sqrt{J_2} = \frac{\sqrt{\frac{1}{2}\left[\left(\sigma_x - \sigma_y\right)^2 + \left(\sigma_y - \sigma_z\right)^2 + (\sigma_x - \sigma_z)^2 + 6\left(\tau_{xy}^2 + \tau_{yz}^2 + \tau_{xz}^2\right)\right]}}{\sqrt{3}}$$

$$\alpha = \frac{2 \cdot \sin\phi}{\sqrt{3} \cdot (3 - \sin\phi)}$$

$$k = \frac{6 \cdot c \cdot \cos\phi}{\sqrt{3} \cdot (3 - \sin\phi)}$$

The characterization of the dam materials was based on triaxial compression tests, permeability tests in the triaxial chamber and grain size analyzes performed on LNEC on rockfill samples type C (see Figure 3).

Sample	σ'_3	**Υ_d (kN/m^3) Preparation**	**Υ_d (kN/m^3) at the end of the consolidation phase**	**Υ_d (kN/m^3) At the end of the test**
EP	100	19.63	19.96	19.24
	200	19.76	20.14	19.73
	400	19.29	19.97	20.04
E1	150	19.36	19.94	19.52
	350	19.34	20.09	21.14
	550	18.78	19.63	19.97
E2	170	19.55	19.89	20.55
	370	20.27	20.66	20.57
	570	20.10	20.59	20.69

Table 1: Dry unit weight (Υ_d) variation in the different test stages.

Laboratory tests performed at LNEC were conducted in three distinct phases. The first tests, in the design stage, were performed on samples obtained in field as well as in samples obtained by fracturing a rock block. During the construction phase of the dam two phases of laboratory tests on samples obtained from quarry work were performed. As expected, there is a trend of increasing density with confining pressure, although with different initial values. Based on the test results, presented in Table 1, and regarding the values of confining pressure of the dam body, it is assumed a value of density for the rockfill material of 20.5 kN /m^3.

Triaxial tests of the consolidated-drained type were performed on 300 mm diameter (T30) cell. Based on the curves of deviatoric stress as function of axial strain, for different confining pressure in the several stages of testing, the Young's modulus value for the axial extension of 0.2% was obtained (Table 2).

	σ_3	$E^{0.2\%}$ (MPa)
EP	100	103
	200	145
	400	34
E1	150	47
	350	200
	550	336
E2	170	126
	370	132
	570	241

Table 2: Young modulus.

The transformed Janbu equation was fitted to the results of triaxial tests, excluding outliers, based on that a graph of the initial elasticity modulus (E_i) with the confinement stress (σ_3), as shown in Figure 4, was generated.

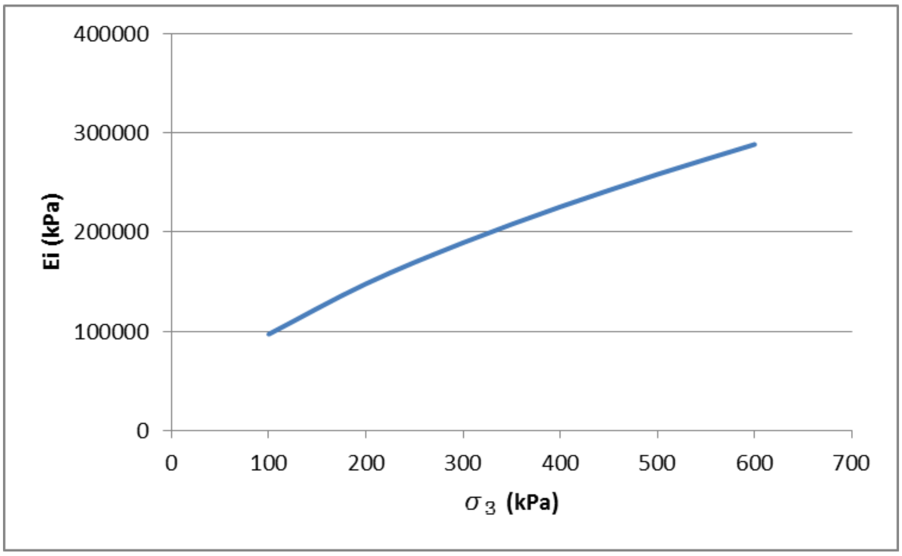

Figure 4: Young's modulus variotion with confining pressure, Janbu's equation.

Knowing that the central zone of the dam, where the largest displacement occurs during the construction phase, shows values of confining pressure around 200 kPa, it is assumed that the value of the Young's modulus to be taken in calculations would be 160 MPa.

Based on the results of the triaxial tests, the friction angle considered was 43.8º, corresponding to a α Drucker-Prager model parameter equal to 0.6. A value of 0.1 for the parameter k was adopted only for numerical reasons.

Hydraulic conductivity of the porous media depends on the degree of saturation. This feature was taken into account by setting the coefficient of permeability given by:

$$\mathrm{k} = \frac{\kappa}{\mu_c} \cdot \rho_c \cdot g$$

where κ is the product of the coefficient of permeability in the saturated state and a factor that takes into account the degree of saturation, which is called permeability reduction factor (r).

Table 3 show the values of the permeability coefficients in the saturated state adopted for the dam materials and the methods or tests used for obtaining it.

Material	k_h (m/s)	k_v (m/s)	Test / Calculation method
Foundation	5×10^{-8}	5×10^{-8}	Lugeon test
Foundation: residual soils	2×10^{-6}	2×10^{-6}	Lugeon test
Rockill, zones C and D	1×10^{-4}	2×10^{-5}	Permeability test in triaxial cell
Rockfill, zones A and B	4×10^{-7}	8×10^{-8}	Kozeny's expression
Grout curtain	1×10^{-7}	1×10^{-7}	Lugeon test (after grouting)
Concrete face	1×10^{-11}	1×10^{-11}	-

Table3: Values and method used to obtain the permeability parameters for the dam materials

The variation in the degree of saturation with capillary pressure can be approximated by a sigmoid function $s_{\mathrm{we}}(p_c; \mu, \varphi) = \frac{1}{1+e^{\frac{-(p_c-\mu)}{\varphi}}}$ where μ and φ are model parameters. The saturation minimal (s_{res}) and maximum values (s_{max}) depend on the particle size of the materials (Figure 5) according with the following expression: $S = s_{\mathrm{we}} \times (s_{\max} - s_{\mathrm{res}}) + s_{\mathrm{res}}$;

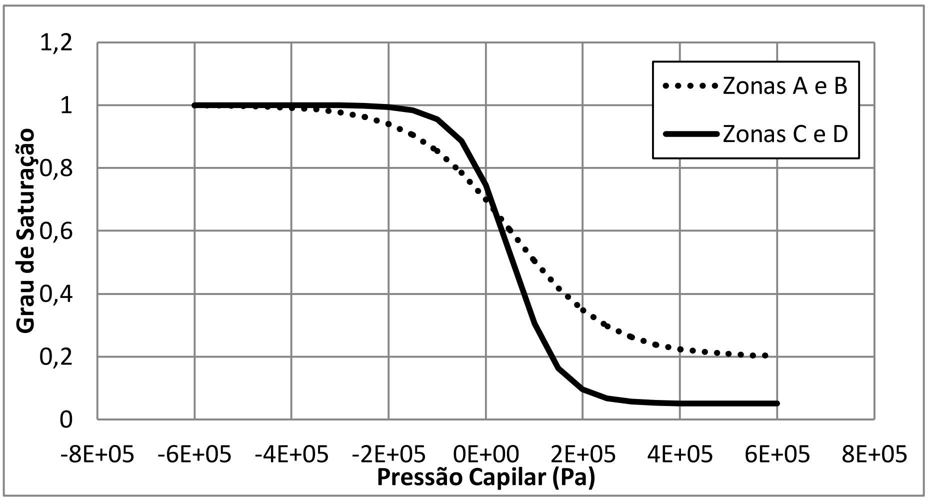

Figure 5: Saturation degree variation with capillary pressure

The variation of the permeability reduction factor (r) of the materials with the degree of saturation is given by a power function with exponent three ($\mathrm{r} = S^3$) or seven ($\mathrm{r} = S^7$), respectively for finer or coarser materials, as shown in Figure 6

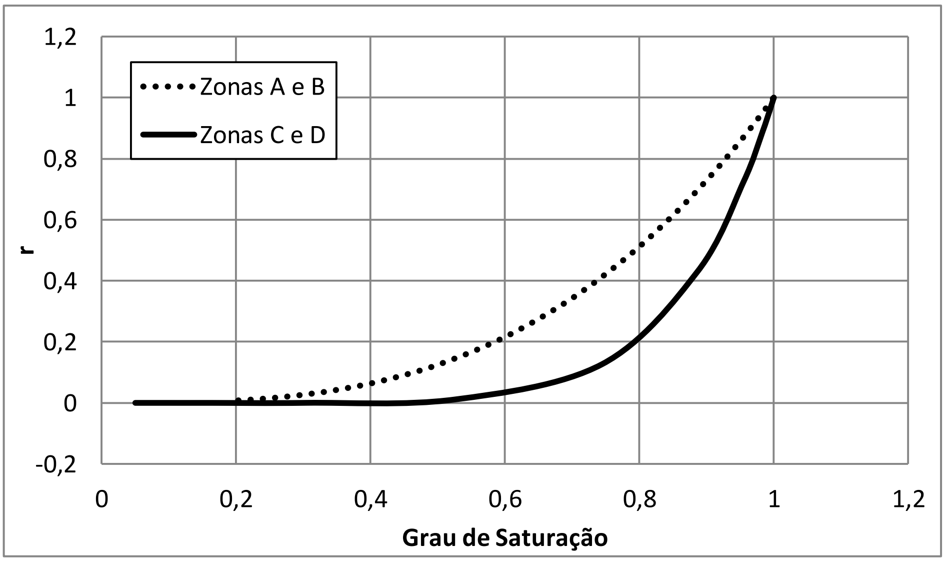

Figure 6: Permeability reduction factor variation with the saturation degree

7 Results and discussion

According to the model and in agreement to other cases, the dam exhibits a low level of deformation during the construction and also during the first filling of the reservoir. Concerning the construction phase the results obtained are in close agreement with the measured values at the dam.

7.1 Construction stage

Relative to vertical displacement, the most significant, the results obtained show that maximum displacements occurs slightly below the central zone of the dam. This behavior was expected because it is a structure built by layers. The maximum vertical displacement obtained in the three-dimensional model is slightly greater than 2 cm, corresponding to a percentage less than 1% of the height of the dam. It is also remarkable the effect of the foundation geometry on the vertical displacements. The maximum values of vertical displacements extend downstream due to local lowering of the foundation. This is a fact to take into consideration, since the differential displacements, even in the downstream region, can reverberate in excessive deformation in the curtain area, as happened in the several case histories.

The displacements in the transverse direction (DY) are symmetrical with respect to the central axis and are in accordance with the normal behavior of the embankment structures. That is, the enlargement of the lower zone of the embankment and narrowing in the upper zone. The vertical and transverse displacements are displayed in Figure 7.

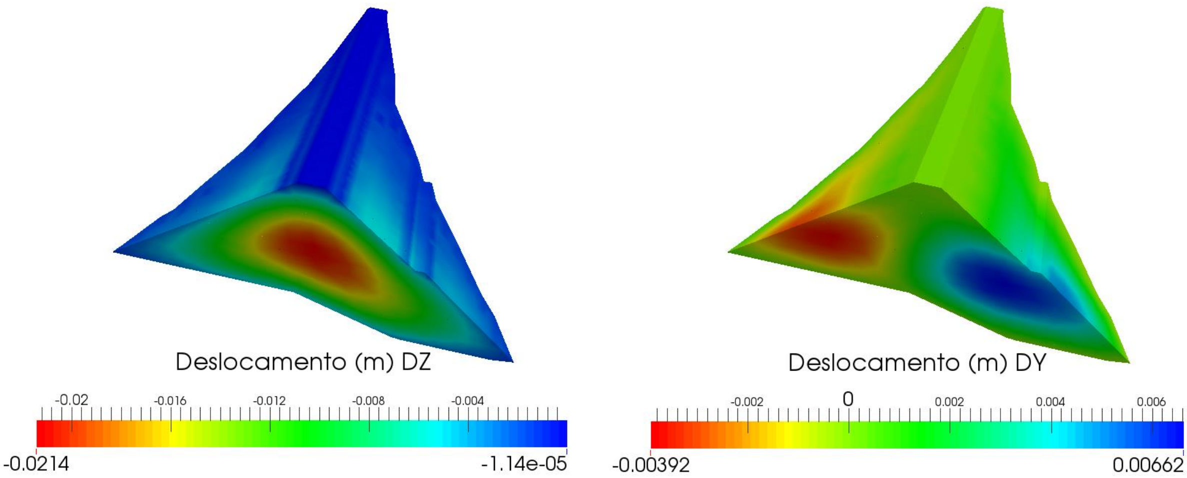

Figure 7: Final stage of construction: (a) vertical displacements; (b) transverse displacements

Figure 8 presents the displacement records in the internal settlement gauges.

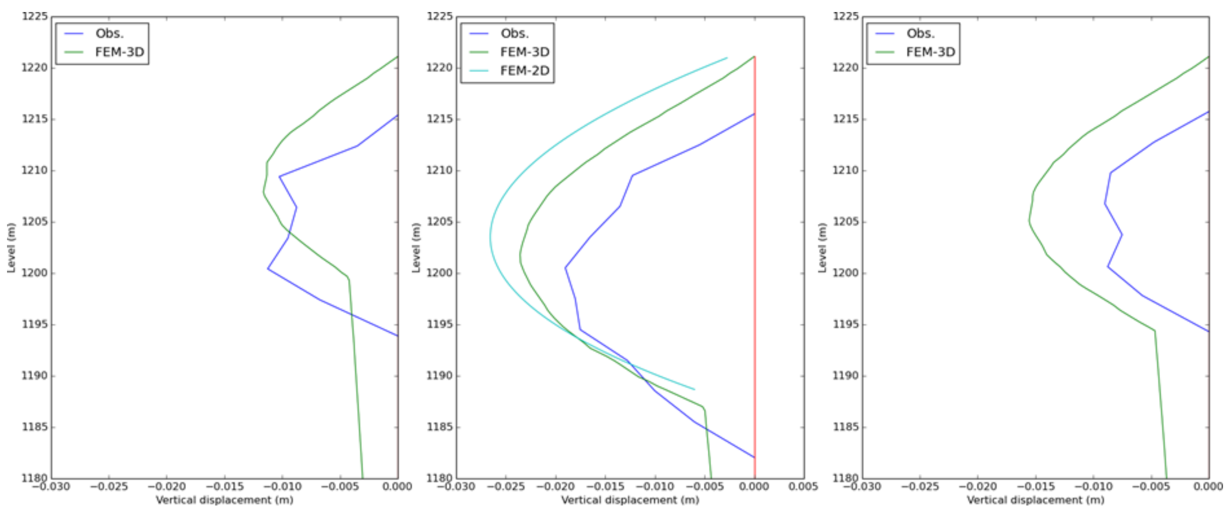

Figure 8: Internal settlements I1, I3 and I5 (measured and calculated)

The diagrams show actual values recorded during the construction and the results from FEM calculations. In the case of gauge I3 (near the highest cross-section) two different calculations are available. The first is from the 3D model and the second is from the 2D model using the same set of mechanical parameters. The recorded values were obtained in August 2014, when the top of the embankment was at elevation 1216.02 m, which corresponds to about 85% of the total height of the dam. Presently the upstream face is being constructed and only after that the final monitoring campaign will be available. The maximum settlement is expected to be in the range of 25 +/- 5 mm.

7.2 First filling stage

The first filling phase is undoubtedly a critical phase in the behavior of CFRD. It is associated with an increased load, relatively fast, in the upstream zone of the dam, materialized by the hydrostatic pressure. This loading induces rising stresses in the upstream area and also an increase of displacement that depends not only on the mechanical characteristics of the material, but also the foundation of geometry and properties

Using the 3D model it is possible to predict the behavior during the first filling (Figure 9). This phase of the loading was performed in 5 steps. Figure 10 shows the displacements in the dam only due to reservoir filling. The following conclusions can be derived: a) the maximum displacement is less than 1 cm, and is expected to occur in the lower third of the dam and near the highest cross-section.

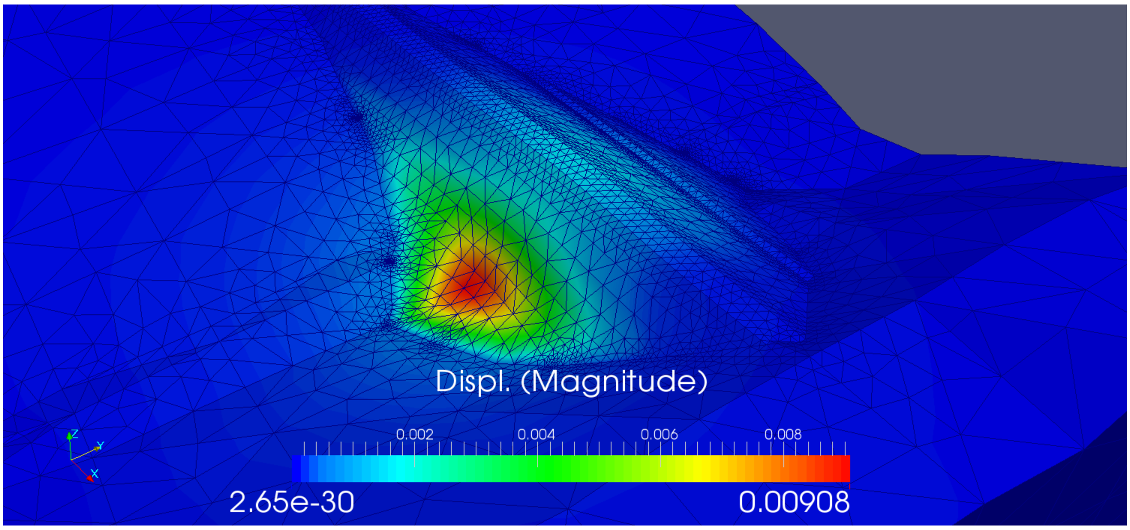

Figure 9: Montesinho dam deformed shape with displacements magnitude due to reservoir filling

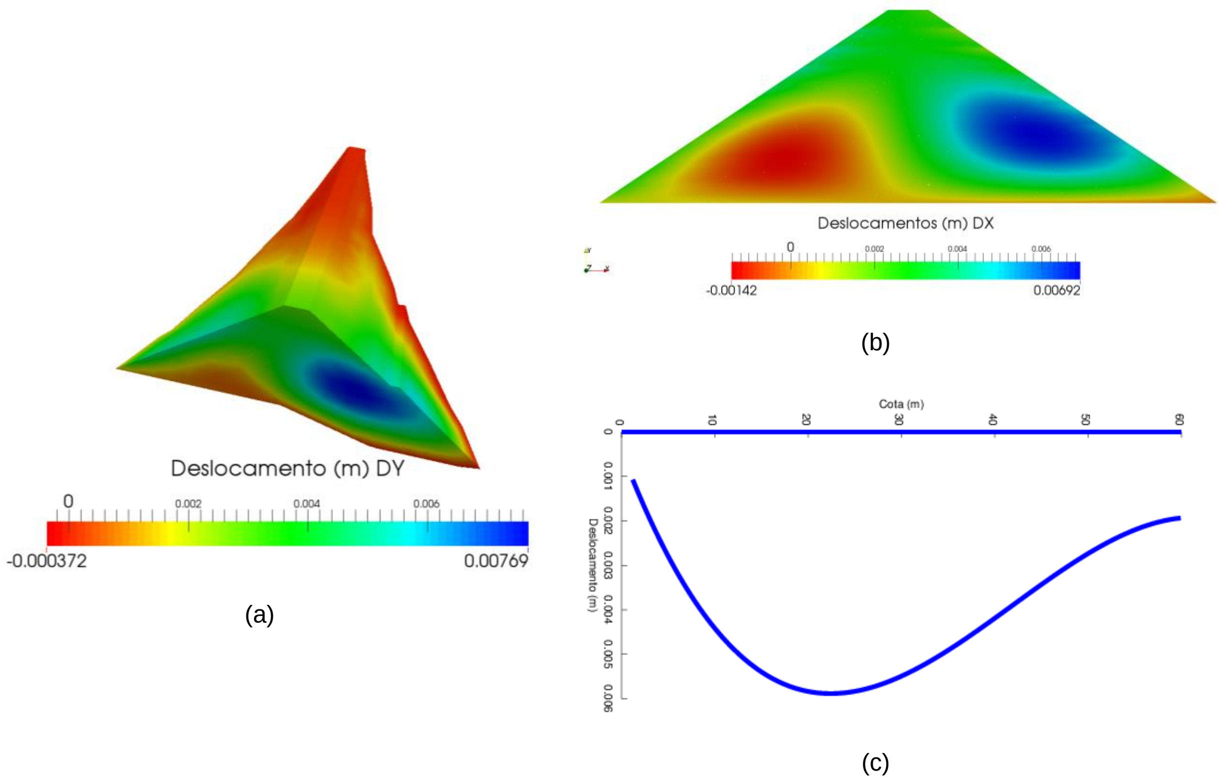

Figure 10: Montesinho dam displacements due to filling: (a) 3D transverse; (b) 2D horizontal (c) 2D normal to concrete curtain

This displacement will be recorded at settlement gauge I4. Near the abutments (where inclinometers I2 and I6 are placed) a lower level of deformation in the range on 2-3 mm is expected. The dam may exhibit an overall downstream movement of about 2 mm near the highest (central) zone.

In the representation of transverse displacements, (Figure 10a)), the effect of filling the reservoir is very representative, because there is a change in the direction of displacement between the final phase of construction and the final stage of filling. At the end of the construction, the embankment is moved out of the dam body, both in the upstream area or in the downstream zone. However, the effect of hydrostatic loading causes in the upstream zone a change of direction in the displacements.

In the two-dimensional model, the results obtained for the offsets in the filling of the reservoir, are similar to the three-dimensional modeling showing, however, higher values of displacement.

7.3 Seepage analysis

The primary objective of this study is the prediction of flows that will be measured during the operation phase of the dam. This represents the sum of flows seeped by the foundation and the rockfill, subtracted from flows that never get to emerge at the surface. This study also aims to, in a very simplified way, predict the effect of hypothetical defects in the curtain will have in seepage flow. Figure 11 presents the seepage velocity magnitude obtained from the calculation with 0 defects in the curtain.

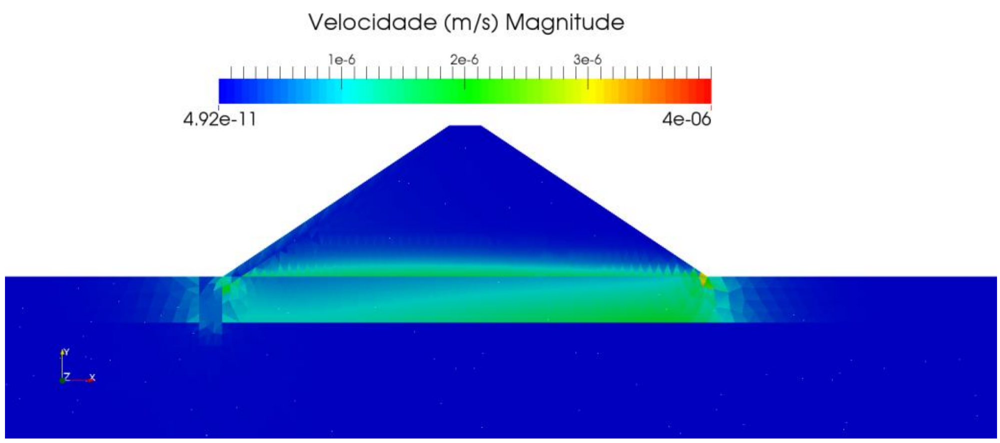

Figure 11: Montesinho dam seepage velocity estimation

The results obtained in the simulations with defects in the curtain, reveal the importance of the curtain and its durability. The predicted flow rate is about 12 times higher than that measured when the curtain showed no defects as shown in Table 4

Curtain defects	Foundation flow (l/s.m)	Rockfill flow (l/s.m)	Flow lost in the foundation (l/s.m)	Measured flow (l/s)
0	0.0105	0.0060	0.00055	1.59
1	0.0580	0.1357	0.00076	19.30
2	0.0553	0.1574	0.00062	21.20

Table 3: Water flow through rockfill and foundation.

8 Final Considerations

8.1 Conclusions

The three-dimensional model of the Montesinho dam shows that the geometry of the foundation is a determinant factor on the stress-strain behavior of the rockfill. In the case study there is an increase of stresses and displacements in the area downstream of the dam axis, where the foundation is at a lower elevation.

The maximum displacements obtained either in three-dimensional models, either in the two-dimensional model occur slightly below the central area of the embankment.

In the three-dimensional model displacements in the longitudinal direction of the dam are observed with the direction of shoulder pads, the area below the downstream berm, and also in some areas within the limits of the shoulder pads. The values obtained in

2D modeling are higher than those obtained in the 3D model both in terms of the level of stresses and the intensity of the displacements.

1.1 Further Developments

The following further developments are suggested:

- Modeling of rockfill embankment zoning;
- Modeling of the rockfill materials through a nonlinear elastic constitutive law, or a more complex elasto-plastic law;
- Modeling the concrete curtain with specific elements;
- Simulation of the behavior of the joints of the dam, using elements of joint.

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