dam world 2018 Third International Dam World Conference
BRAZIL • Foz do Iguaçu • September 17-21, 2018

SETTLEMENT ANALYSIS OF THE MONTESINHO CFRD

João Manso*, **João Marcelino**[†] and Laura Caldeira[‡]

* Laboratório Nacional de Engenharia Civil (LNEC)
Av. do Brasil, 101, 1700-066 Lisboa, Portugal
e-mail: jmanso@lnec.pt, webpage: www.lnec.pt

Keywords: CFRD, rockfill, FEM, settlements

Abstract. *Concrete face rockfill dams (CFRDs) are becoming a widely used type of rockfill dam all over the world. Until recently, the design and construction of CFRDs were empirically based primarily on precedent and engineering judgments. Few numerical or analytical methods have been developed to properly evaluate the deformation of CFRDs, which is important for dam safety and for subsequent evaluation of seismic performance. This paper describes the setup of a finite element method (FEM) model for the three dimensional simulation of the construction of Montesinho CFRD and filling of the its reservoir using the Code-Aster code. The prototype of the study is the 36.5 m high Montesinho CFRD, located in the north of Portugal near the Spain border, in the Sabor river in the Montesinho Natural Reserve, witch has been completed in 2016. Settlements are regarded as a key indicator of dam safety. Therefore, the time-dependent settlement behaviour of Montesinho CFRD is studied on the basis of in situ settlement-monitoring records. The monitoring results covered the construction period, the initial filling of the reservoir and 2 years of operation. Two different elastoplastic models (Druker Prager, Cam Clay) were implemented to better model the rockfill materials. The model parameters were calibrated by large-scale triaxial tests, performed on materials used in the dam. The numerical results agree well with in situ monitoring records of dam settlements, allowing to predict the deformation of the dam in the future.*

1 INTRODUCTION

Concrete-faced rockfill dams (*CFRD*) are a major type of rockfill dams, whose structure consist of cushion, transition, main rockfill, and secondary rockfill zones. Due to their good adaptability to topography, geology and climate, use of locally available materials, cost-effectiveness, simple construction and short construction period, they have been repeatedly constructed in recent decades, in some cases higher than 200 m.

The major concerns regarding the design and operation of these structures are related to the deformations of rockfill zones and to the stresses in concrete slabs and slab joints. Numerical methods, such as finite element method, can be used to predict dam deformation during construction and operation. However, the reliability of the results for the adopted model depends significantly on its suitability to model rockfill materials.

Several methods have been adopted for the modelling of rockfill. The nonlinear elastic Duncan-Chang E-B model has been widely used to model *CFRD* construction, due to the simplicity and clear physical meaning of its parameters. Xing et al.[1] studied the mechanical and hydraulic properties of weak rockfill during placement and compaction in three different dam projects. The stresses and the strains in the dams were evaluated using

[†]Laboratório Nacional de Engenharia Civil (LNEC)
[‡]Laboratório Nacional de Engenharia Civil (LNEC)

a two-dimensional finite element software, where they implemented the nonlinear hyperbolic model Duncan-Chang E-B, and compared to the field measurements. Zhou et al.[2] also applied this model to analyse the measured deformations resulting from continuous monitoring of the Shuibuya *CFRD*. They performed a displacement back-analysis for parameters using a hybrid generic algorithms (*HGA*s), allowing the prediction of the long-term deformation of the dam. Their simulations were performed in two-dimensional plain-strain conditions, considering the time-dependent deformation, the construction process and water storage. They showed that the settlement rate decreased with time and tended to stabilize, and also that the material's deformation modulus was smaller than those obtained from the corresponding laboratory tests. Once again they showed that the parameters in Duncan-Chang E-B model could be evaluated using a group of conventional triaxial tests.

Li and Desai[3] developed a finite element procedure for stress-deformation analysis of dams, including sequential embankment construction, seepage analysis (including transient and steady free surface), and a combination of both. They modelled the mechanical behaviour using linear elastic, nonlinear or piecewise linear elastic (hyperbolic) and plasticity (Drucker-Prager) models. They adopted the same mesh for both seepage and for the stress analysis and conveniently superimposed the two effects. Providing the proper conditions, it was possible to incorporate effects of partial saturation during construction and to include changes in the dam geometry during the deformation process. This procedure provided satisfactory correlation with analytical solutions and field observations, which could be useful for nonlinear stress, seepage and stability analysis of dams. Xu et al.[4] modified the generalized plasticity model for sand, which was based on the work of Pastor and Zinkiewicz[5] and Ling and Liu[6], in order to describe the behaviour of rockfill materials, particularly their unique pressure dependency due to particle crushing. Then they incorporated the model into a three-dimensional *FEM* program and simulated the construction process and reservoir impounding of the Zipingpu *CFRD*, comparing the numerical results to field measurements. This proposed numerical procedure does not consider the creep deformation of rockfill, nor the influence of very large particles on rockfill behaviour, and thus may underestimate dam deformation and slab stresses, particularly during the period of reservoir operation. However, these underestimations were somehow compensated by the use of a lower-density rockfill material to calibrate the model parameters, due to the laboratory apparatus dimensions.

In this paper a three-dimensional *FEM* model was developed. The Drucker-Prager and the Cam Clay models were incorporated into the *FEM* program and simulated the construction, first filling and operation phases of the Montesinho *CFRD*, returning a reasonable consistency between the predicted and monitoring results. This will allow to analyse the behaviour of the dam until now and inferred the response during its operation.

2 MONTESINHO CFRD

The Montesinho dam is located in the Sabor river, in the Montesinho Natural Reserve. It is a *CFRD* built to provide water supply to the city of Bragança, 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 with 7 m of width. The total volume of the embankment is of about 174 000 m^3 and consists of granitic rockfill obtained from the quarries located upstream of the dam in the reservoir area.

The reservoir as a capacity of 3.69 hm^3 (net volume of 3.53 hm^3) with a flooded area of 35.8 ha and a catchment area of 10.1 km^2. The normal water level is at 1217.50 m (above sea level – a.s.l.) and the maximum water level is at 1219.73 m (a.s.l.). The freeboard has 1.37 m, therefore the crest is located at level 1221.10 m (a.s.l.). The upstream and downstream embankments originally had a slope of 1:1.5 (v:h), however, during the construction, it was decided to include a berm in the downstream shell. Table 1 presents the general characteristics of Montesinho dam.

Dam detail	Value
Crest height	36.5 m
Crest length	310.0 m
Crest width	7.0 m
Normal water level (a.s.l.)	1217.50 m
Maximum water level (a.s.l.)	1219.73 m
Crest elevation (a.s.l.)	1221.10 m
Dam slope upstream	1 : 1.5
Dam slope downstream	1 : 1.5
Total embankment volume	174×10^3 m^3
Total reservoir area	35.8 ha
Catchment area	10.1×10^6 m^2
Total reservoir volume	3.69×10^6 m^3
Usable reservoir volume	3.53×10^6 m^3

Table 1: General characteristics of Montesinho dam and reservoir.

From a geological viewpoint, in the dam site and the reservoir area the outcropping blocks and top layers of the bedrock consisted essentially of a two-mica granite with coarse grain. The rock presented a generalized and mild to medium kaolinization of the feldspars, often corresponding to a weathered granite (*W3* and *W4*), with low mechanical resistance and a low deformability modulus. At greater depths its quality increased (*W2* to *W3*).

Spread over the foundation existed a thin coating of top soil or organic soil which has been removed prior to the construction of the dam. Figure 1 and 2 present the plan and the cross-section of Montesinho dam. As it can be seen from the figures, the valley is asymmetrical with an average slope of 1:6.5 (v:h) above level 1200 m (a.s.l.), on the right bank, and 1:2.6 (v:h) on both left and right banks, below that level.

3 STRUCTURE MONITORING SYSTEM AND SETTLEMENTS ANALYSIS

The technical characteristics of the dam, combined with the geological formations of its foundation and the surrounding area, have led to the adoption of a detailed monitoring system. It encompasses a variety of geotechnical and geodetic instruments and techniques capable of monitor the deformation of the Montesinho dam. The field instrumentation placed in the dam body included settlement gauges (capable of recording vertical and horizontal interior displacements) distributed in three important cross-sections, 1–1, 2–2 and 3–3, with cross-section 2–2 being the major monitoring cross-section of the dam (Figure 2). The concrete slab deformations were measured by three inclinometers installed in the cushion zone, at the same cross-sections: 1–1, 2–2 and 3–3. Surface deformations at the crest of the dam were measured by 12 monitoring gauges separated from each other of about 25 m. Its positions are shown in Figure 1.

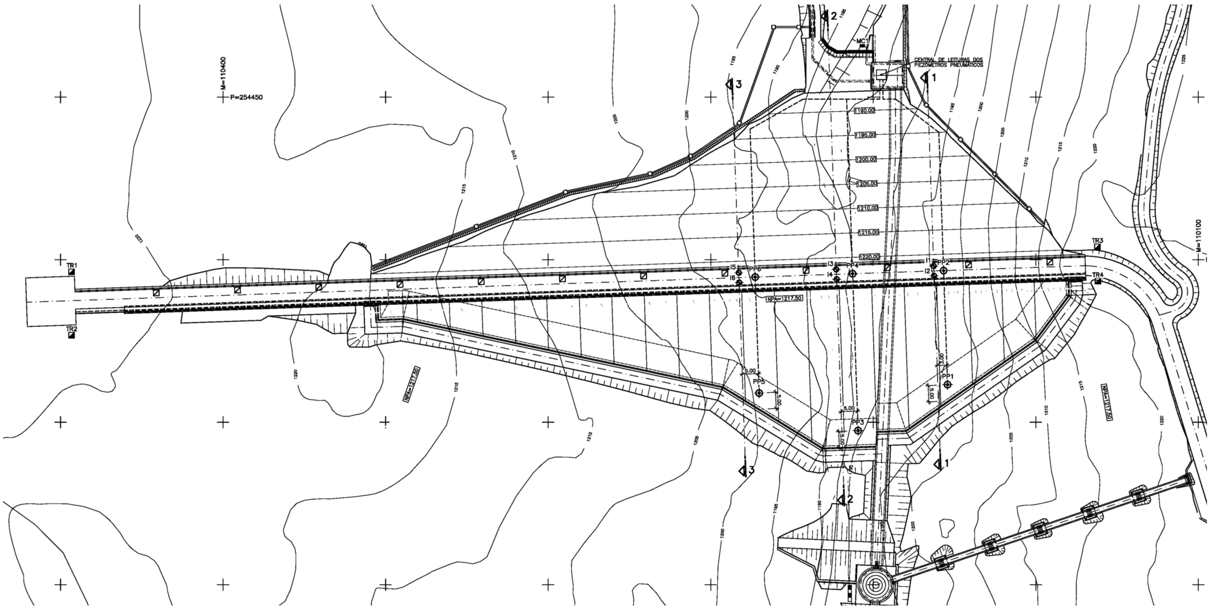


Figure 1: Plan of Montesinho dam.

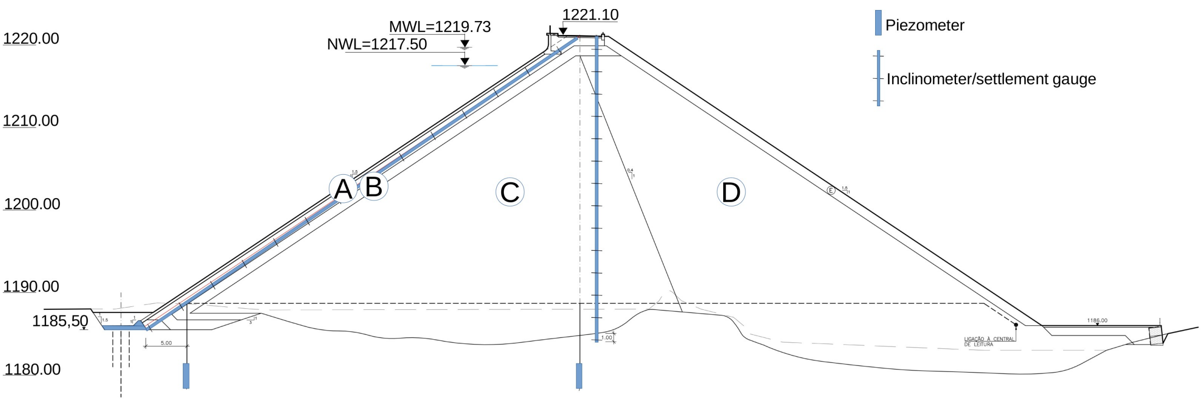


Figure 2: Layout of settlement gauges in cross-section 2-2 of the Montesinho dam.

The monitoring system also included 3 sets of 2 piezometers in the foundation of the dam. In each set, the first piezometer is located immediately after the grout curtain while the second is located near the dam axis. The purpose of this system is to evaluate the curtain efficiency.

Finally, to complement this system a total flow measuring system, near the toe of the dam is also included. During the construction of the dam regular measurements of the internal vertical displacements and water pressures in the foundation were made. The results of the former are presented below in comparison with the *FEM* model.

In addition to geotechnical monitoring, periodic dam inspections and geodetic surveys have been performed since the construction period. These inspections included weekly routine visual inspections and monthly main inspections performed by engineers and technical staff.

3.1 Dam embankment deformation

As expected[2, 7], vertical displacements in the embankment dam developed faster during the construction phase and then decreased progressively in time, during the stage of the first filling of the reservoir. The settlements monitoring system relied on different settlement gauges displayed through the body of the rockfill dam.

To better understand the progression of the settlements of the Montesinho *CFRD*, three major periods were analysed:

- Construction (before May, 2015);
- First filling of the reservoir (May, 2015, until March, 2016);
- Operation (from March, 2016).

The internal settlements of the rockfill embankment were recorded at three important cross-sections, 1–1, 2–2 and 3–3, from 22 April, 2014, to 19 May, 2015, (construction period) and from 19 May, 2015, to 6 July, 2016 (first filling and operation). Figure 3 presents the internal settlements registered at cross-section 2–2 (highest cross-section) for both periods. The accumulative settlements registered reached the value of 27 mm at the level 1200.5 m (a.s.l.) at the end of construction (Figure 3a). The displacement profile resemble a parabolic shape[8, 9], where the maximum was observed approximately at middle height, corresponding to 0.07 % of dam's height.

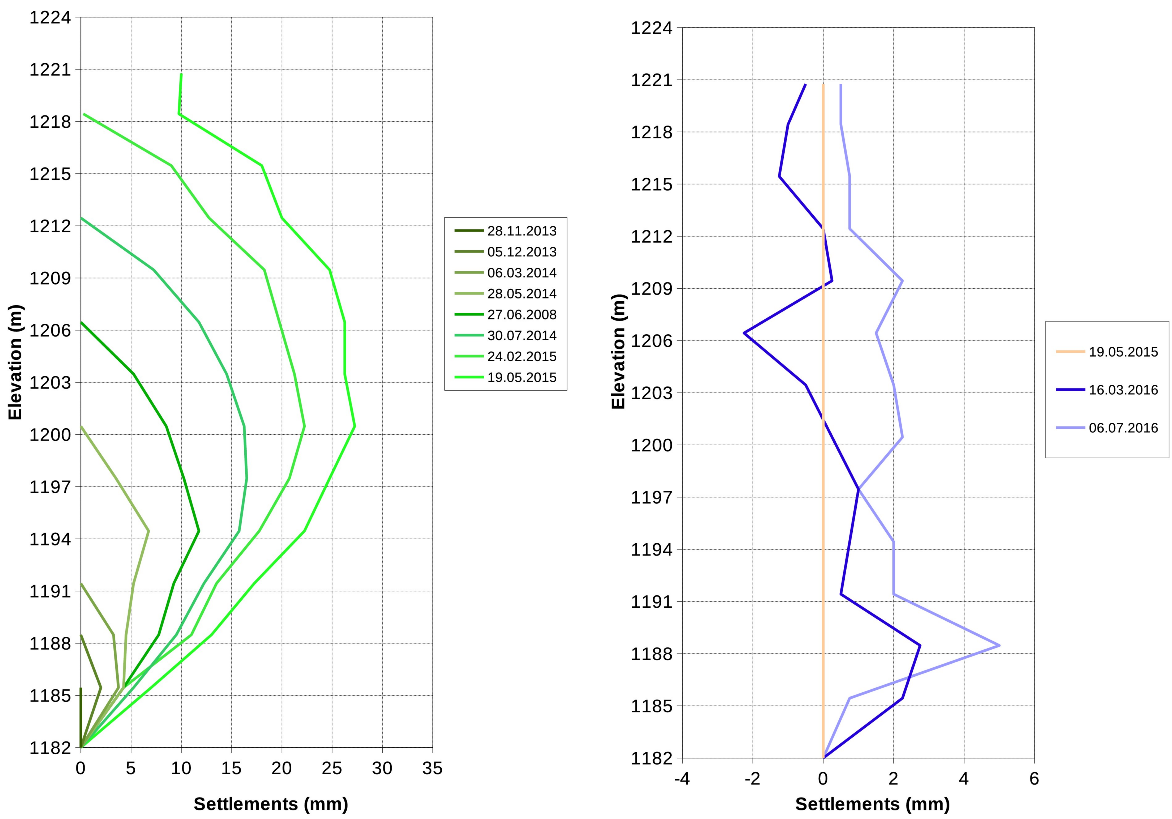
The reservoir impounding was planned to occur into two parts. It began on 8 September, 2015, with the water at level 1196.08 m, quickly increasing to the first level at 1210.19 m on 5 November, 2015 (2 months). The second part of the impoundment began on 11 February, 2016, with the water at level 1210.40 m, increasing until level 1217.69 m, on 29 February, 2016 (18 days). To better understand the embankment deformation during the reservoir impounding, a new reference was considered since 19 of May, 2015 (Figure 3b). The additional settlements were 5 mm (level 1188.5 m) or 0.01 % of dam's height. It can be concluded that the construction phase had a more pronounced effect, on embankment settlements, than the immediate impounding loading.

Figure 4 presents the evolution of the internal settlements (in relation to the settlements registered on 19 of May, 2015) and compare them with the reservoir level, for the first filling and operation periods. The settlements were analysed for level 1220 m at cross-sections, 1–1, 2–2 and 3–3. It can be seen that during this period the settlements slightly varied and no clear tendency was observed, between the embankment settlements and the reservoir level.

It should be mentioned that the errors that result, from using this method to monitor internal settlements in the embankment, are of the same order of magnitude of the settlements registered. Therefore, due to the dimensions of the embankment of the Montesinho *CFRD*, their settlements are expected to be small, which could explain some of the registered swelling and irregularities.

3.2 Lateral displacements registered in the embankment and in the concrete face

As mentioned previously, the concrete slab plays the important role in *CFRD*s as the impervious element. Any crack in the slab would reduce the integrity of the seepage control system and weaken the structure, which, in the worst case scenario, may threat the safety of the dam, but most often only compromise its functionality. Nonetheless, it is the deformation of the rockfill that influences the deformation of the concrete face slab. If rockfill experiences excessive deformation after the construction of the concrete face slab, it will cause the separation between the slabs and the cushion layer or can even lead to slab cracking[2]. Therefore, in order to reduce this potential risk in *CFRD*s, the study of the rockfill deformation plays an important role to make the rockfill compatible with the concrete face. This will decrease the number of cracks and eventually improve the design of *CFRD*s.



a) Construction period.

b) First filing and operation (in relation to the end of construction).

Figure 3: Internal settlements registered in the embankment at settlement gauge nº3.

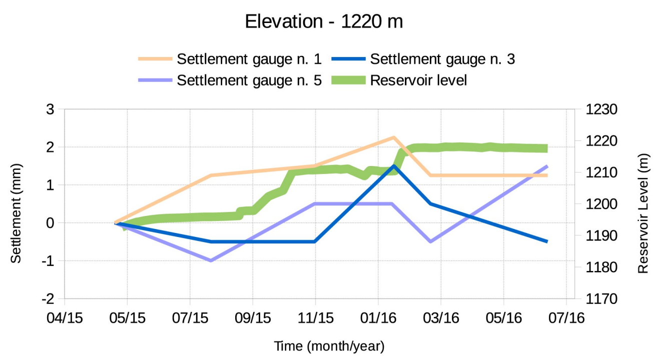
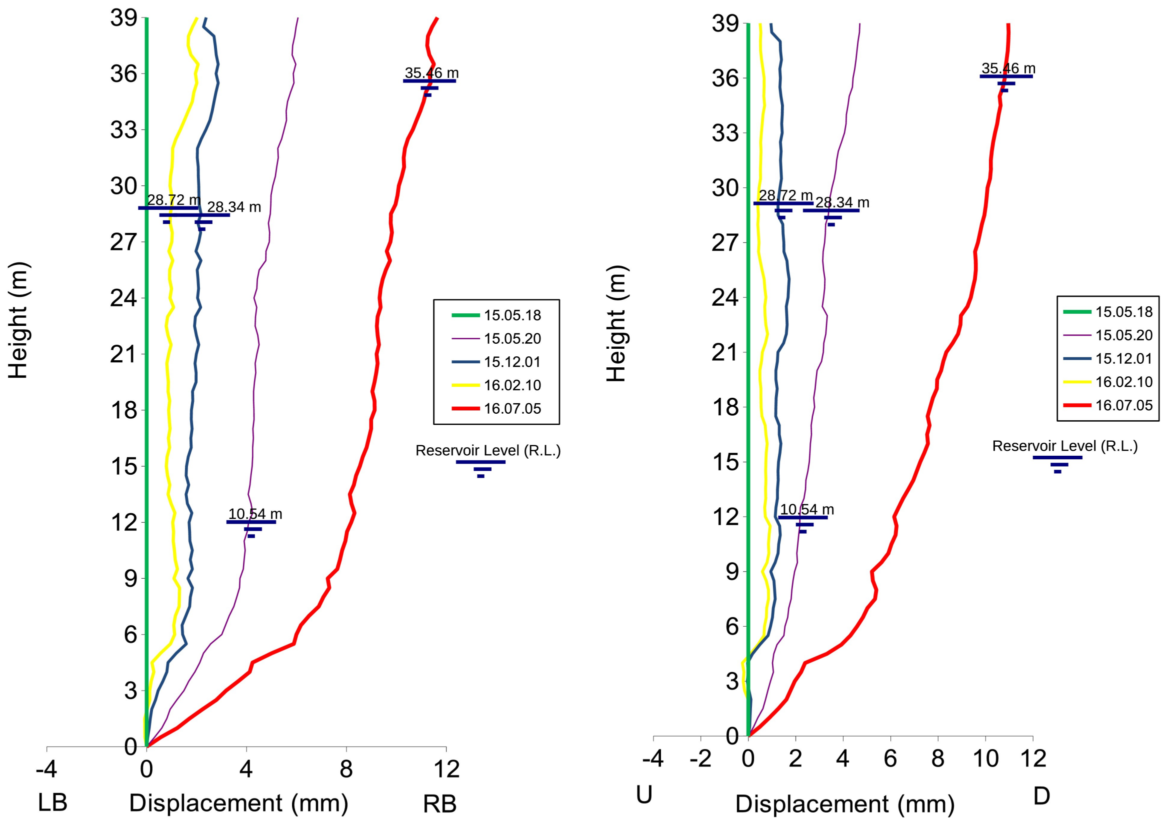


Figure 4: Internal settlements registered in the embankment during first filling and operation.

The monitoring of the rockfill embankment can be an effective method to characterise and analyse deformation. It has also proven to be very helpful for warning of some abnormal behaviour of the dam, as well as for understanding its deformation mechanism[2, 7].

Figure 5 shows the cross-valley horizontal and upstream-downstream displacements registered in the embankment, for cross-section 2–2. The inclinometers are anchored into the bedrock. The maximum cross-valley horizontal displacement was equal to 11.6 mm (5 July, 2016), at 39 m above the base of the inclinometer toward the right abutment. It was obtained during the dam operation.



a) Left-right abutment.

b) Upstream-downstream.

Figure 5: Lateral displacements registered in the embankment from May 2015 to July 2016. Vertical inclinometer nº 3.

The upstream-downstream maximum displacement registered was equal to 11.0 mm (5 July, 2016), toward the downstream direction. The maximum was obtained at 39 m above the base of the inclinometer located in cross-section 2–2, during operation.

Figure 6 shows the upstream-downstream displacements registered in the concrete face, for the cross-section 2–2. As it can be seen, the inclinometer registered displacements mainly toward the downstream direction. In the beginning of the first filling, it registered a maximum displacement of 4.7 mm at level 41 m above the base of the inclinometer (downstream direction, 20 May, 2015), whereas the maximum displacement registered during operation occurred at level 42.5 m, with a value of 6.5 mm (upstream direction, 5 July, 2016).

All results considered, it can be concluded that the rockfill embankment deformation is compatible with the concrete face. The displacements observed in the concrete face were considerable acceptable and may lead to a reduced number of cracks in the slab. Therefore, its impervious capabilities were guaranteed.

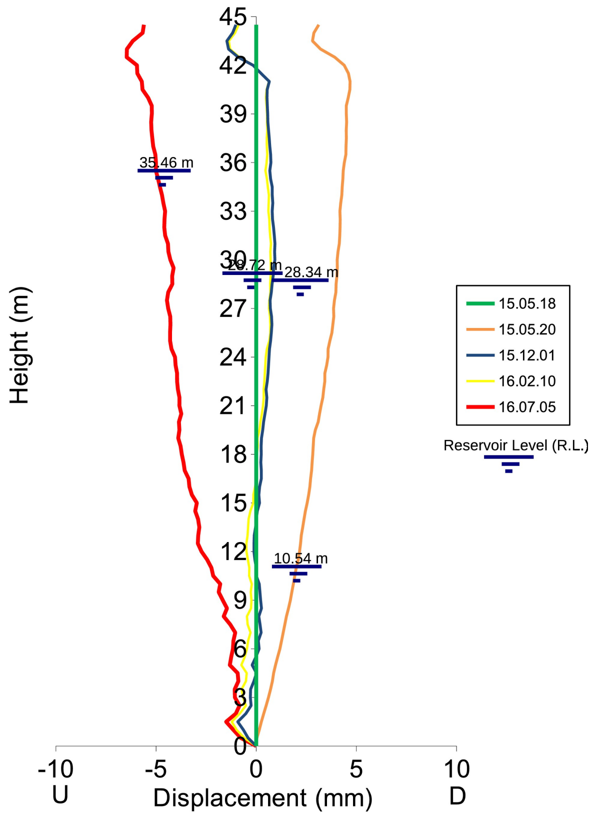


Figure 6: Lateral displacements registered in the concrete face from May 2015 to July 2016. Inclined inclinometer nº 6.

4 FINITE ELEMENT ANALYSIS

The structural analysis software *Code-Aster* was used to perform the *FEM* elasto-plastic analyses on the Montesinho *CFRD*. *Code-Aster* is an open source software package for civil and structural engineering, developed as an in-house application by the French company Éectricité de France (*EDF*). In October 2001, it was released under the terms of the *GNU* General Public License. *Code-Aster* runs on GNU/Linux workstations or clusters.

Using *Code-Aster*, a model of the Montesinho *CFRD* and its dam site area was prepared in order to study the deformations and the stress-strain behaviour of the dam. Three basic models have been analysed based on three major periods of the dam: its staged construction, the first filling of the reservoir and the dam operational lifetime.

4.1 Geometry setup

After analysing the results of the structure monitoring, it is necessary to compare them with the observed deformations obtained from *FEM*. This comparison will allow to verify the structure's behaviour and the adopted design parameters.

The geometry of the model of Montesinho *CFRD* was divided into two parts (Figure 7): the foundation and the dam, and was performed using a python script written by Marcelino et al.[10]. The foundation was generated from a set of blocks with a triangular base and top. The foundation surface was obtained from a simple point sampling of the dam foundation surface and then, those points were submitted to a Delaunay triangulation, creating the block set. After all blocks had been created, they were fused together originating the geometry of the foundation. The *FEM* model extends 103.0 m in the upstream and 50.8 m in the downstream directions, whereas the depth of the model is almost thrice the maximum height of the dam (102.4 m).

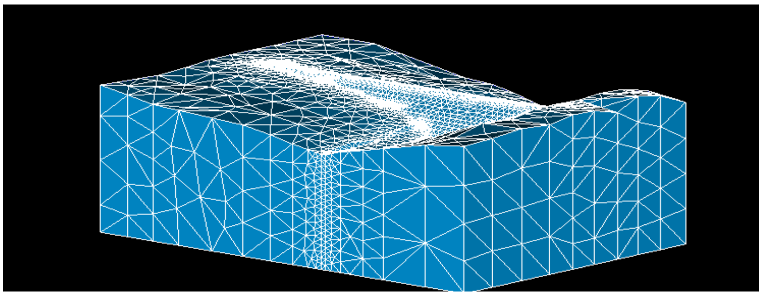


Figure 7: Montesinho *CFRD* numerical model

On the other hand, the dam geometry was highly parametrized in the script. The user had to specify two points from the crest (the upstream edge) and all the relevant features of the dam geometry, such as, the crest, the bank width, the slopes, ... Then, the dam was generated as a regular solid and cut by the foundation, using a boolean cut geometric operation. By doing this it was possible to adjust the dam geometry to the surface of the foundation. The final generated blocks were independent, but compatible.

In this study, the numerical modelling was performed under three-dimensional conditions. Since the stress and strain paths were relevant to the rockfill behaviour, the construction of the dam was made in layers[11]. On the other hand, in order to correctly account for the deformation of the dam being built, the model needs to consider the displacements of the existing layers, not the one being placed. Therefore, the dam model was also split in layers by means of several cutting planes, allowing for an arbitrary user-defined number of layers[10].

4.2 Finite element mesh generation

The *FEM* mesh was generated using the simplest algorithm available in the software (*Netgen*) and all the default hypothesis in the *Mesh module*. In the case of Montesinho *CFRD* model, the generated mesh consisted of approximately 17K nodes and 83K tetrahedra elements. From those, about 30K elements were used to build the 15 layers of the dam itself. In order to allow the definition of the boundary conditions and the construction sequence, groups of nodes, faces and elements were defined in this module.

As mentioned previously, the simulation of the construction of the dam was made in layers, to account for the construction deformations, but also because the behaviour of rockfill material depends on the stress (and strain) paths. Following the Naylor proposal[12], to correctly account the deformations during construction, the displacements of a newly constructed layer were dismissed. With this method the model was capable of having null displacements in the crest, when the dam was completed, since in reality each layer is always built up to the designed level.

Time was not considered in the material behaviour, so a constant construction rate was adopted, dividing the embankment in equal layers.

The *FEM* analysis considered the combination of two types of loads to model the first filling of the reservoir: gravity load and water pressure. The first one was applied by assigning the unit weight of the materials. To model the stages of first filling of the reservoir and the operating time of the Montesinho *CFRD*, two alternatives were considered. The first one consisted of modelling the water as special incompressible finite elements, whereas the second one consisted of modelling the water load applying surface loads. Considering all the implications of the former hypothesis in the *FEM* model, particularly a larger mesh, it was decided to adopt the second alternative. The static water pressure was applied as a surface load on the upstream embankment, in accordance with the reservoir level.

4.3 Parameters identification

4.3.1 Drucker-Prager failure criterion

Marcelino et al.[10] developed a *FEM* model to simulate the construction process of Montesinho *CFRD* and its first filling. As a first approach, the behaviour of the rockfill material was modelled using an Drucker-Prager failure criterion.

Drucker and Prager (1952) have proposed this criterion as a smooth approximation to the Mohr-Coulomb law. It is similar to the von Mises criterion in which an extra term is included to introduce pressure-sensitivity[13]. The onset of plastic yielding occurs when the equation:

$$\sigma_{eq}+\alpha I_I - R(p)=0 \tag{1}$$

is satisfied, where σ_{eq} represents a function of the deviatoric effective stresses, σ', $I_I=tr(\sigma)$ represents the trace of the effective stresses, α represents a pressure-sensitive coefficient:

$$\alpha=\frac{2\sin(\varphi)}{3-\sin(\varphi)} \tag{2}$$

where φ represents the friction angle, and $R(p)$ represents a function of the cumulative plastic deformation:

$$\text{If} \quad 0<p<p_{ult} \quad \text{then} \quad R(p)=hp+\sigma_y \tag{3}$$

$$\text{If} \quad p\geq p_{ult} \quad \text{then} \quad R(p)=hp_{ult}+\sigma_y \tag{4}$$

where h represents the hardening internal variable, p_{ult} represents the ultimate cumulative plastic deformation and σ_y represents the yielding stress which is given by:

$$\sigma_y=\frac{6C\cos(\varphi)}{3-\sin(\varphi)} \tag{5}$$

where C represents the cohesion coefficient. The parameters of the Drucker-Prager failure criterion adopted in the model are presented in Table 2.

Material type	Behaviour	Parameter		Value
Foundation	Linear elastic	E	[Pa]	$15 \cdot 10^7$
		ν		0.2
		ρ	[Kg/m3]	2194
Rockfill	Drucker-Prager	α		0.5
		σ_y	[N]	10
		p_{ult}	[N]	0.2
		h		$7 \cdot 10^7$

Table 2: List of material properties for the Drucker-Prager failure criterion.

4.3.2 Cam Clay model

The formulation of the original Cam Clay model as an elasto-plastic constitutive law was developed at the University of Cambridge by Roscoe and his coworkers[14]. Later, Roscoe and Burland[15] proposed the modified Cam clay model. Since then the Cam Clay model has been widely applied in geomechanics and its numerical implementation has been dealt quite extensively. Although this model was developed to simulate clay behaviour, several authors have used it[16] or developed models in terms of its elastoplastic framework[17] to model rockfill behaviour. Cam-Clay model has useful potential to describe rockfill behaviour.

Once again, considering the experimental data of the rockfill from Montesinho, it was possible to determine the parameters of this formulation. Table 3 lists the adopted values, where E represents the Young's modulus, ν is the Poisson's ratio, ρ is density of the material, μ is an elastic Poisson's ratio, λ represents the slope of the normal compression line, κ is the slope of the swelling line, Γ is the specific volume on the critical state line at p' = 1 kPa., M represents the critical state line in q':p' space.

Material type	Behaviour	Parameter		Value
Foundation	Linear Elastic	E	[Pa]	$15 \cdot 10^7$
		ν		0.2
		ρ	[Kg/m3]	2194
Rockfill	Cam-Clay	μ		0.25
		λ		$4.84 \cdot 10^{-3}$
		κ		$1.03 \cdot 10^{-3}$
		Γ		1.259
		M		1.64

Table 3: List of material properties for the Cam-Clay model.

The developed FEM models were used to analyse the construction process and the first filling of the Montesinho dam.

4.4 Embankment deformation during construction

After the construction of each layer of the rockfill embankment, the settlements were determined. They were evaluated in cross-section 2–2, that was presented before. The monitoring results date from 19 May, 2015, corresponding to the final period of construction and beginning of the first filling.

The behaviour of the embankment is characterised by a low level of deformation during the construction, which is in close agreement with other embankments found in the literature and other embankments monitored during construction. Figure 8 presents the displacement records in the vertical internal settlement gauges, for monitoring results and *FEM* analysis for both formulations. The differences obtained in the values of settlements may result from neglecting the creep deformation and the effect of particle size and void ratio (in the determination of the model parameters). Nonetheless, numerical results show a similar trend to that of the field measurement results. The maximum settlement registered in gauge I3 (near the higher cross-section) was equal to 27.0 mm. It can be seen that the simulated settlements, at the same level of the embankment, were 23.7 mm (Cam-Clay model) and 21.3 mm (Drucker-Prager model). Since the Cam-Clay formulation returned slightly better results regarding the embankment deformation, this formulation was adopted in the rest of the paper to analyse the behaviour of the dam.

Figure 9 shows the deformation contours of cross-section 2–2, where the settlements were increased 1000-fold. It can be seen that the maximum settlements were obtained at the middle height of the embankment. Although the *FEM* model returns relatively smaller settlements than the ones obtained with field measurements, they are in accordance with similar rockfill works and, predictably, the safety and serviceability of the dam is assured for the expected loads.

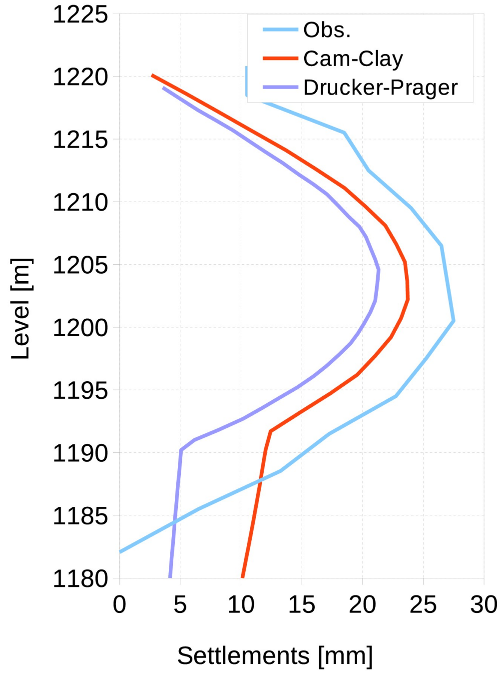


Figure 8: Internal settlements. Registered and observed in gauge I3.

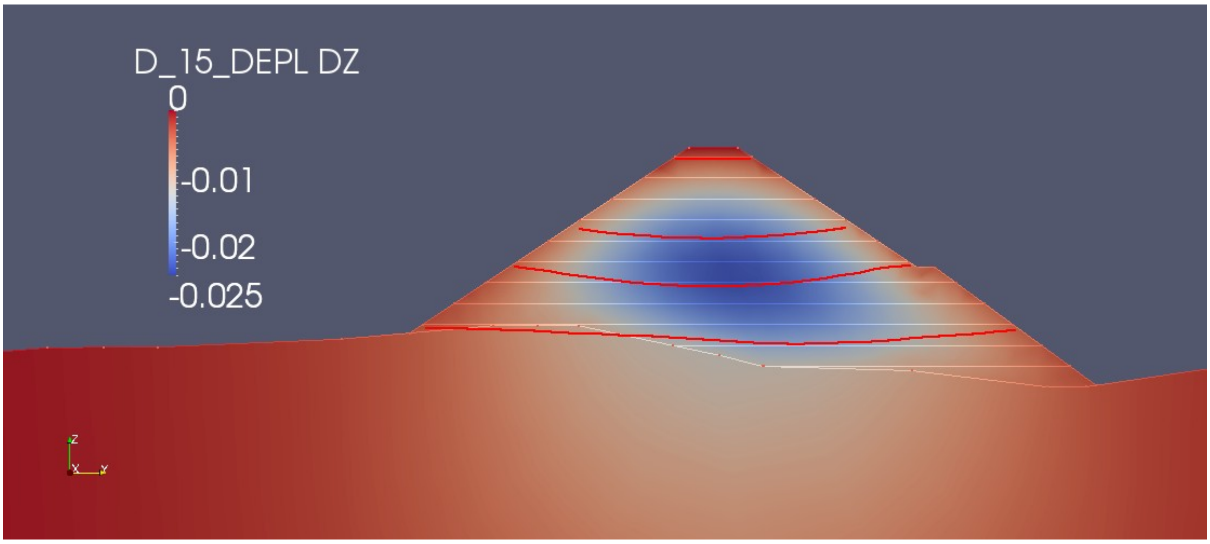


Figure 9: Internal settlements calculated at different levels. Cross-section 2–2.

4.5 Embankment deformation during first filling

The same numerical model was used to predict the behaviour of the *CFRD* during the first filling. This phase of the loading was divided in 5 steps. The stress and deformation states obtained during embankment construction were adopted as the initial state in this analysis, not considering the effect of creep. Figure 10 shows the settlements in the dam for full storage (water level 1217.5 m). The maximum displacement calculated is less than 1 cm, obtained in the middle of the dam, near the highest cross-section. Near both abutments, the level of deformation in expected to be less than 2 mm, for cross-sections 1–1 and 2–2.

Figure 11 presents the estimative of the horizontal displacements, in the upstream-downstream direction, where the dam exhibits an overall downstream movement of about 1 cm, near the central cross-section. On the other hand, the model predicted right-left abutment displacements between 3–5 mm, presented in Figure 12.

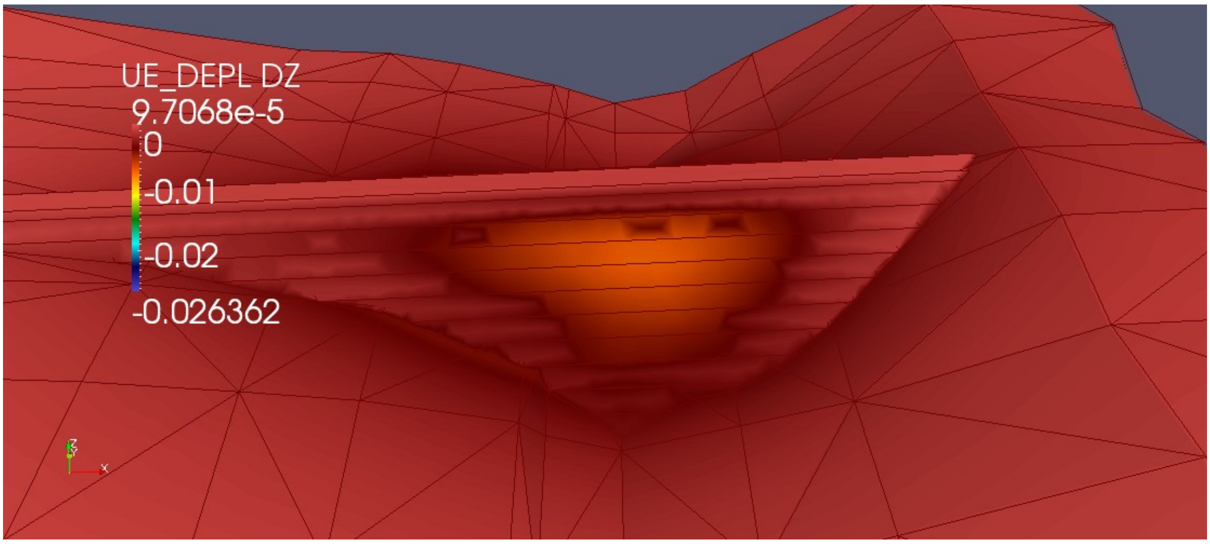


Figure 10: Settlements due to the first filling.

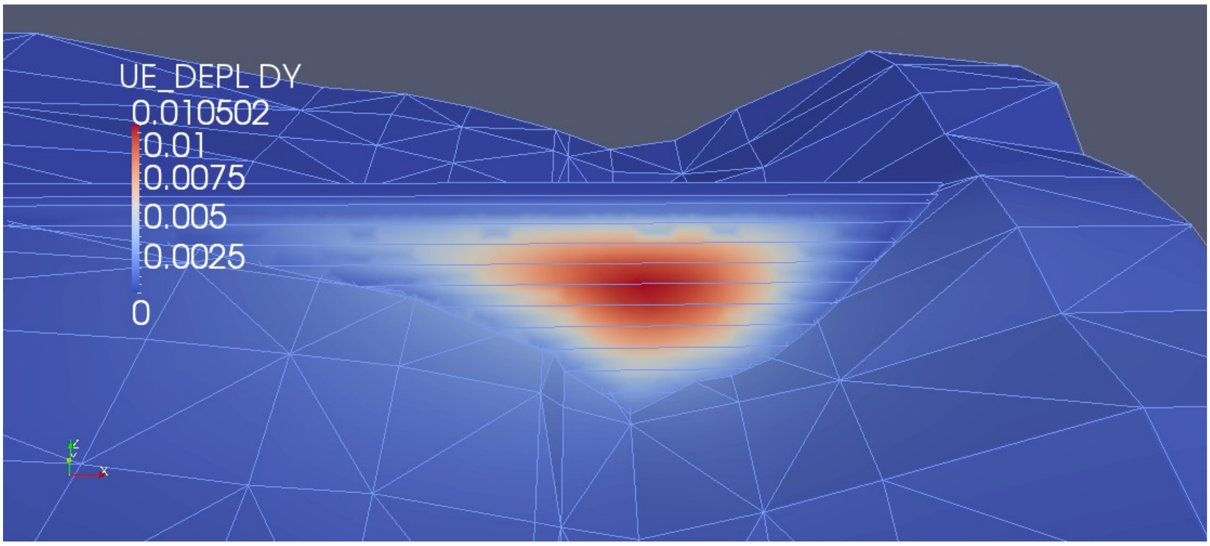


Figure 11: Upstream-downstream displacements.

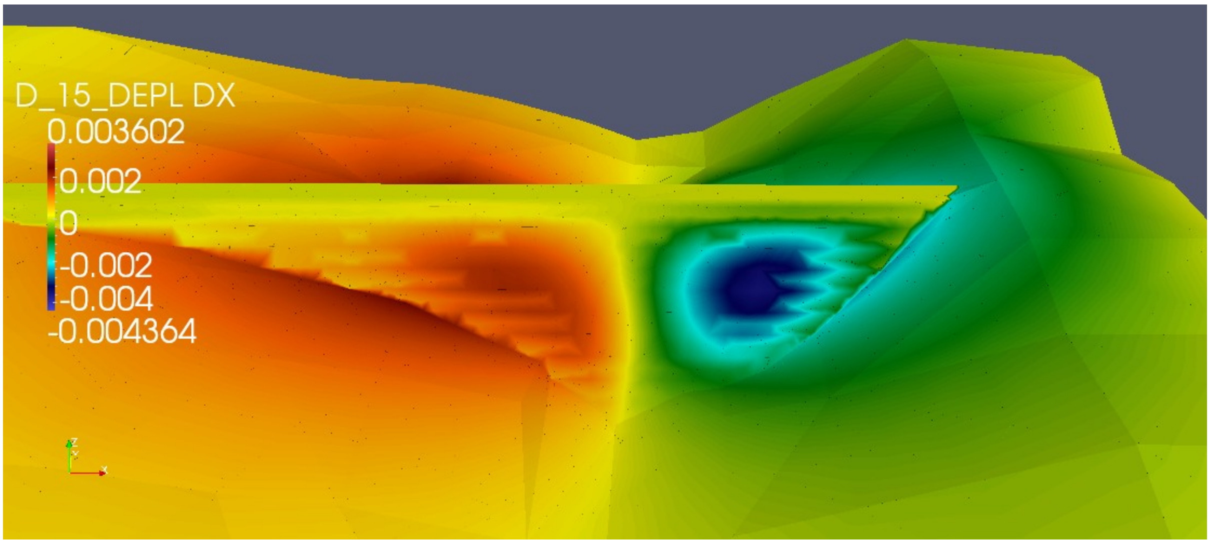


Figure 12: Right-left abutment displacements.

5 CONCLUSIONS

The monitoring of geotechnical structures, such as *CFRD*'s, is very important to access its serviceability and safety. During the construction of Montesinho *CFRD* a set of monitoring equipments were installed to control deformation of its rockfill body. Posteriorly, monitoring records of settlements at the crest and inside the dam body allowed to analyse the behaviour of the rockfill embankment. This analysis showed that the settlements were acceptable, for this type of structure, and that their maxima occur at the central cross-sections of the embankment, around middle height.

After the construction of the Montesinho *CFRD* was completed, the analysis of the behaviour of the rockfill embankment, during the first filling of the reservoir, showed that the settlement rate decreased and tended to stabilize, comparing with the period of construction.

The *FEM* model, based on the experimental results, proved capable of properly modelling the deformability of the rockfill used in the Montesinho *CFRD*, for both formulations (Cam-Clay and Drucker-Prager). Then, the behaviour of the rockfill embankment were numerically simulated during the construction and reservoir filling processes. The simulated settlements trends and values at the monitoring points were consistent with the field measurements, proving the accuracy of the proposed numerical procedure, and that it can be used to analyse the embankment behaviour.

It can be concluded that the Montesinho *CFRD* settlements were be very small, and that its deformation is essentially stable. Therefore, the construction techniques and the monitoring system installed to control the dam deformation are successful and it is predicted that the behaviour of the dam will stabilise, considering the expected loads acting upon the structure, and serviceability problems are not expected (in normal situations).

To sum up, a *3D FEM* model can be capable of faithfully predicting embankment settlements and general dam behaviour, during any phase of a dam (construction, first filling and operation). The validation of such numerical model, or assessment of a constitutive model, requires high-quality experimental data collected from a carefully executed testing programme. Such data may be obtained from full-scale field testing or laboratory-based research.

REFERENCES

[1] H.-F. Xing, X.-N. Gong, X.-G. Zhou, and H.-F. Fu, “Construction of Concrete-Faced Rockfill Dams with Weak Rocks,” *J. Geotech. Geoenvironmental Eng.*, vol. 132, no. 6, pp. 778–785, 2006.

[2] W. Zhou, J. Hua, X. Chang, and C. Zhou, “Settlement analysis of the Shuibuya concrete-face rockfill dam,” *Comput. Geotech.*, vol. 38, no. 2, pp. 269–280, 2011.

[3] G. C. Li and C. S. Desai, “Stress and seepage analysis of earth dams,” *Manager*, vol. 109, pp. 946–960, 1983.

[4] B. Xu, D. Zou, and H. Liu, “Three-dimensional simulation of the construction process of the Zipingpu concrete face rockfill dam based on a generalized plasticity model,” *Comput. Geotech.*, vol. 43, pp. 143–154, 2012.

[5] M. Pastor, O. Zinkiewicz, and H. Chan, “Generalized plasticity and the modelling of soil behaviour.,” *Int. J. Numer. Anal. Methods Geomech.*, vol. 14, no. August 1989, pp. 151–190, 1990.

[6] H. I. Ling and H. Liu, “Pressure-Level Dependency and Densification Behavior of Sand Through Generalized Plasticity Model,” *J. Eng. Mech.*, vol. 129, no. 8, pp. 851–860, 2003.

[7] V. Gikas and M. Sakellariou, “Settlement analysis of the Mornos earth dam (Greece): Evidence from numerical modeling and geodetic monitoring,” *Eng. Struct.*, vol. 30, no. 11, pp. 3074–3081, 2008.

[8] L. Pagano, A. Desideri, and F. Vinale, “Interpreting settlement profiles of earth dams.,” *J. Geotech. geoenvironmental Eng.*, vol. 124, no. 10, pp. 923–932, 1998.

[9] A. Gurbuz, “A new approximation in determination of vertical displacement behavior of a concrete-faced rockfill dam,” *Environ. Earth Sci.*, vol. 64, no. 3, pp. 883–892, May 2011.

[10] J. Marcelino, A. Serrano, J. Manso, L. Caldeira, and J. Paixão, “3D Analysis of Montesinho CFRD using code-aster FEM program.,” in *Second International Dam World Conference*, 2015, pp. 1–13.

[11] D. J. Naylor, G. Pande, B. Simpson, and R. Tabb, *Finite elements in Geotechnical Engineering*, vol. 6, no. 4. 1981.

[12] D. J. Naylor, G. Pande, B. Simpson, and R. Tabb, *Finite elements in Geotechnical Engineering*, vol. 6, no. 4. 1981.

[13] E. A. de S. Neto, D. Peric, and D. R. J. Owen, *Computational methods for plasticity - Theory and applications*. 2008.

[14] K. H. Roscoe, “Mechanical behaviour of an idealized wet clay.,” in *3rd Eur. Conf. Soil Mech.*, 1963, pp. 47–54.

15] K. H. Roscoe and J. B. Burland, “On the generalised stress-strain behaviour of ‘wet’ clay.,” in *Eng. plasticity.*, Cambridge.: Cambridge Univ. Press., 1968, pp. 535–609.

16] J. Manso, “Predicting the behaviour of rockfill embankments,” Instituto Superior Tecnico, Universidade de Lisboa, 2017.

17] L. A. Oldecop and E. E. Alonso, “A model for rockfill compressibility,” *Géotechnique*, vol. 51, no. 2, pp. 127–139, 2001.