Seepage and stability analyses of a zoned earth dam subjected to variable water heads: numerical simulations with Abaqus

Nicola Pontani, Chiara Rossignoli & Donatella Sterpi
Department of Civil and Environmental Engineering, Politecnico di Milano, Milano, Italy

Cristina Jommi
Department of Geoscience and Engineering, Delft University of Technology, Delft, The Netherlands
Department of Civil and Environmental Engineering, Politecnico di Milano, Milano, Italy

ABSTRACT: The behaviour of a zoned earthen dam is analysed by means of a finite element model implemented in Abaqus. The analysed dam was built in Slovenia at the end of the 1980s for irrigation and flood protection purposes. In 2007, detection of a wet stain on the downstream slope forced the drawdown of the reservoir to carry out remedial works. Transient fully coupled hydro-mechanical analyses under partially saturated conditions have been conducted to investigate the behaviour of the dam from its construction to present. Available in-situ measurements have been exploited to calibrate the model and to assess the reliability of the predictions. To address the effects of the wet stain and of the remedial works on the dam body, stability analyses have been performed at significant times. Future scenarios have been analysed, using provided information on the time schedule of the reservoir impounding after completion of the remedial works. The results show that the high recorded pore water pressures inside the dam body may be explained by damage experienced by the irrigation pipelines and that remedial works are necessary to restore the dam functionality.

1 INTRODUCTION

Embankment dams represent over 80% of dams built in the world (Wrachien & Mambretti 2009) and approximately 40% of all embankment dam failures have been attributed to internal erosion due to uncontrolled seepage through the dam body (ICOLD 2017). During construction and operation, embankments are in a partial saturation condition, and they are subjected to various hydro-mechanical loads following from consolidation and from time variations of river water level. To properly account for all the aspects that contribute to the response of the system, coupled hydro-mechanical numerical analyses are mandatory. The development of the numerical model should be supported by appropriate monitoring data. The comparison between numerical results and site observations will validate the approach and will help in the calibration of the model to predictive purposes. The Formulators proposed the present benchmark to assess how different modelling assumptions and approaches could influence the results.

To simulate this benchmark, 2D finite element numerical analyses have been run with Abaqus 6.23 (Dassault Systèmes 2021). Fully coupled hydro-mechanical analyses in a three-phase porous media (Zienkiewicz et al. 1990) have been conducted to study the problem.

The focus has been on explaining the cause of a wet stain, detected on the downstream slope of the dam during regular maintenance in 2007, after twenty years of operation. Back analyses have been performed to calibrate the numerical model (case 1), where missing information was managed on engineering judgement. The comparison between numerical results and monitoring data at relevant time instants has allowed validating the model and has suggested including a defect in the pipe system (case 2). Eventually, the future state of the dam has been analysed when subjected to the given hydraulic load history (case 3).

2 MODEL DESCRIPTION

2.1 *Geometry*

The dam investigated in the benchmark is a zoned earth dam, with a clay core. Figure 1 shows the domain assumed for the analyses: the geometry of the dam has been defined based on the schematic 2D cross-section provided by the Formulators (Zvanut et al. 2022), while the dimensions of the foundation layer have been chosen as 245x30m. In the same figure, the finite element mesh considered in the analyses is shown. The numerical solution has been obtained using fully coupled pore pressure-displacement elements with reduced integration (CPE8RP).

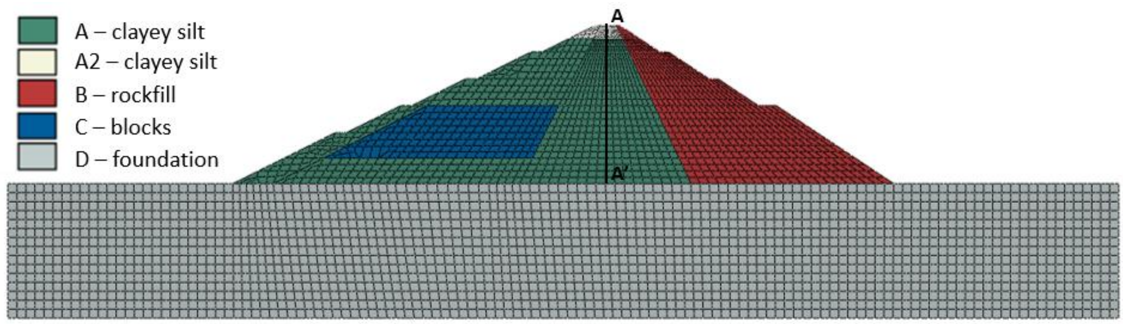

Figure 1: 2D geometry considered in the analyses; different colours represent the 5 materials considered in the model. Section A-A' represents the central vertical axis of the dam, where significant quantities will be analysed in the following.

2.2 *Material behaviour and properties*

The elastic-perfectly plastic Mohr-Coulomb model has been used to model the behaviour of the different materials. Mechanical and hydraulic properties have been defined combining information provided by the Formulators with results of back-analyses. The values of the relevant parameters adopted in the numerical simulations are listed in Table 1, where ρ_d is the dry density, c and ϕ are the shear strength parameters, ψ is the dilatancy angle, E is the Young modulus, ν is the Poisson ratio and k is the saturated hydraulic conductivity. The calibration stage has focused on the clay core material A. Drained values of shear strength parameters have been assumed starting from information on the nature and the state of the soil, while stiffness and hydraulic conductivity have been calibrated as described in the following paragraphs. To include the unsaturated response, water retention curves and hydraulic conductivity functions have been defined for fine and coarse materials, thus providing the relations between suction, degree of saturation and hydraulic conductivity, as shown in Figure 2. The choice of the specific relationships adopted in the analysis has been based on literature studies (Alonso et al. 2005, Caruso & Jommi 2005, Rossignoli & Sterpi 2021).

Table 1. Material properties after the calibration.

Zone	ρ_d [kg/m^3]	c [kPa]	ϕ [°]	ψ [°]	E [MPa]	ν [-]	k [m/s]
A	1578.0	10.0	30.0	20.0	11.0	0.30	10^{-8}
A2	1894.0	36.0	36.0	24.0	7.0	0.40	10^{-6}
B	2267.0	0.0	38.0	25.0	37.0	0.30	10^{-3}
C	2267.0	0.0	38.0	25.0	37.0	0.30	10^{-4}
D	2407.0	32.0	39.0	26.0	62.0	0.25	10^{-9}

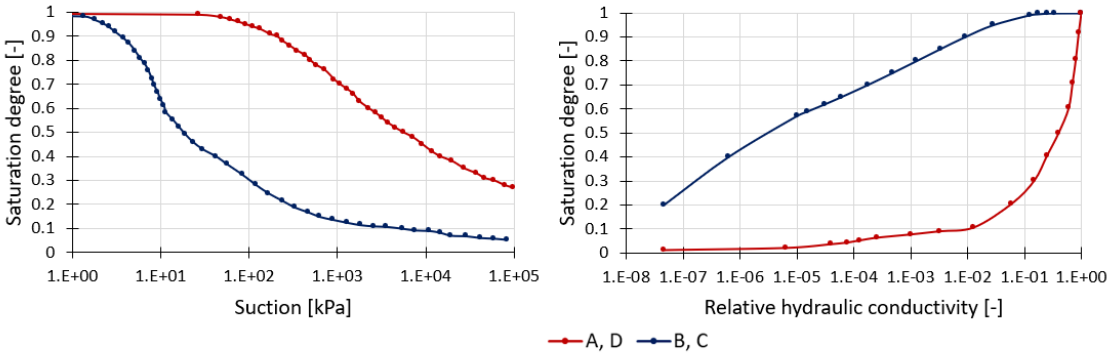

Figure 2. SWRCs (on the left) and permeability functions (on the right) assumed for fine A and D (red line) and coarse B and C (blue line) materials.

2.3 *Time dependent seepage analyses*

The reservoir water level at the upstream side of the dam is modelled as a time-dependent boundary condition on the pore-pressure field. The recorded water levels as well as the approximate time-history simulated in the analyses are represented in Figure 3.

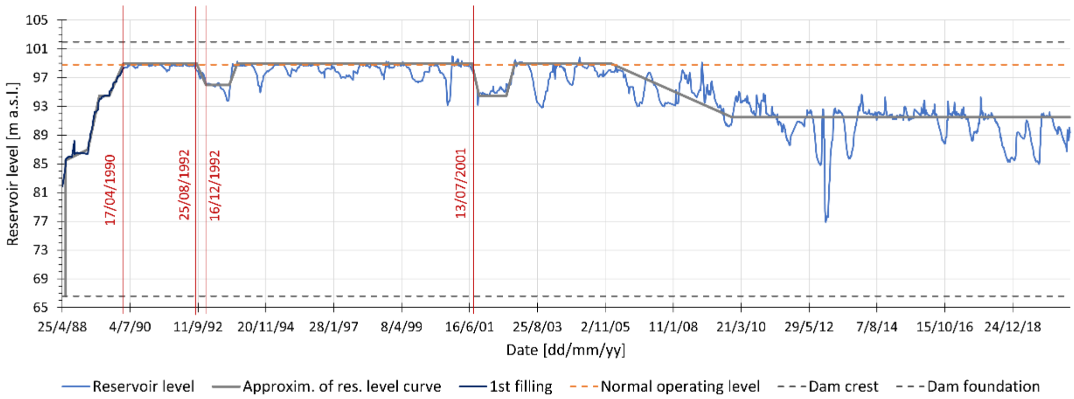

Figure 3. Time evolution of reservoir level from 1988 to 2020: recorded values (blue line) and approximate time-history simulated in the analyses (grey line). Significant dates indicated with red lines refer to Case1-Task2.

2.4 *Stability analyses*

The stability analyses described in the following have been conducted according to the Shear Strength Reduction (SSR) method (Matsui & San 1992, Griffiths & Lane 1999). It consists in reducing c and ϕ until slope failure occurs, with the failure mechanism of the slope identified by localisation of shear strains. According to this method, the Safety Factor FS plays the role of a shear strength reduction ratio and is evaluated according to Equation 1:

$$FS = c/c_r = \tan\phi/\tan\phi_r \qquad (1)$$

where c_r and ϕ_r are the reduced shear strength parameters.

3 RESULTS AND DISCUSSIONS

3.1 *Case 1*

Case 1 simulates the construction of the dam, the first filling of the reservoir as well as its operating conditions before leakage is detected. Numerical results are compared with field monitoring data and the stability of the dam is checked at significant time instants.

3.1.1 *Task 1: dam construction*

At the beginning of the analysis, the water table coincides with the ground surface and the geostatic effective stress state is assigned to the foundation layer. The construction of the dam is reproduced by a gradual growth of the gravity loading, maintaining the same water table level. Due to capillary rise, this condition leads to partial saturation of the dam body, as shown in Figure 4. To obtain realistic settlements of the whole dam body, the stiffness of material A has been calibrated as E=11 MPa. Figure 5 reports the displacements at the end of the construction, along the vertical section A-A’, represented in Figure 1. The maximum settlement of about 0.80 m is observed slightly below the dam crest.

Stability of both the downstream and upstream slopes of the dam before the first filling of the reservoir is evaluated by means of the SSR method. Since the instability mechanism observed downstream involves materials with high cohesion, at this stage the FS of the downstream slope is greater than the one of the upstream slope (Tab. 2). Figure 6 reports the contours of deviatoric plastic strains, which indicate the predicted failure surfaces in the two cases.

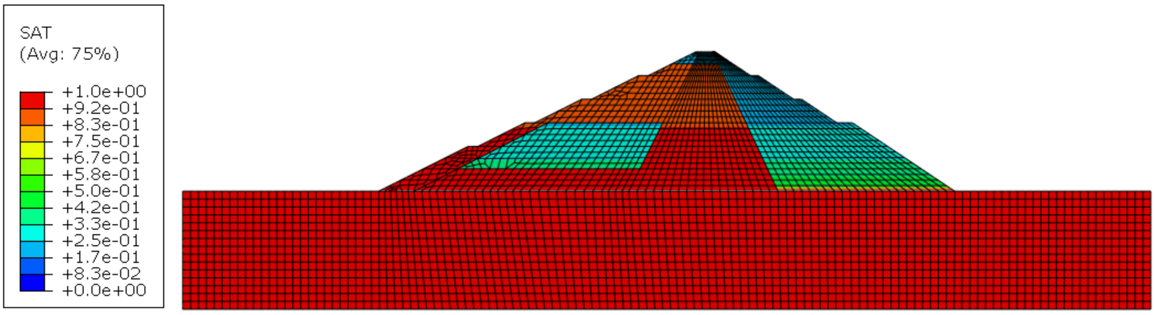

Figure 4. Contour of the degree of saturation at the end of the construction.

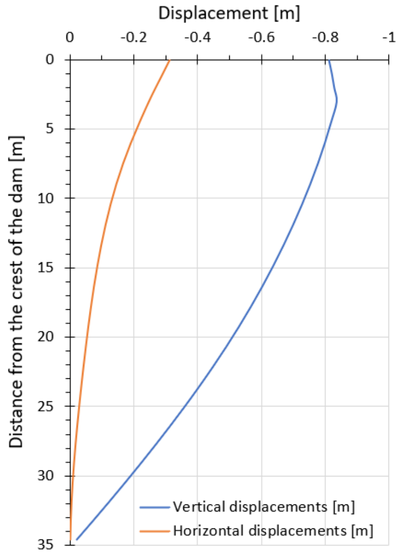

Figure 5. Vertical and horizontal displacement along direction A-A' at the end of the construction.

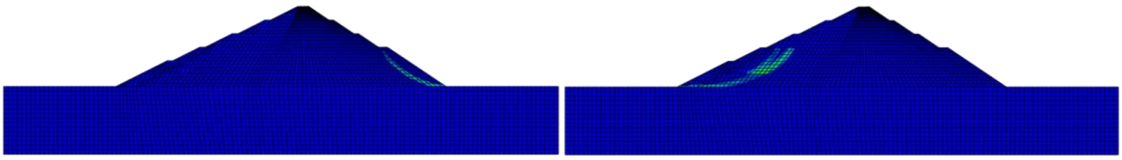

Figure 6. Contour of the plastic deviatoric strain showing the sliding surfaces at upstream (on the left) and downstream (on the right) slope.

3.1.2 *Task 2: first filling and operating conditions*

The simulation of the construction stage aims at reproducing the state of the dam before the filling of the reservoir, started on 25/04/1988 and lasted for 722 days. To simulate the initial conditions of the system, the water level is increased from ground surface to the position indicated as the beginning of the filling, at around 82 m a.s.l. Since there is no information about the duration of this phase, a steady-state analysis is performed.

Starting from this situation, the water level in the reservoir is gradually raised, following the temporal evolution shown in Figure 3. Coupled hydro-mechanical analyses under transient conditions are performed, to predict the variations in time of the stress – strain state as well as the pore water pressure regime within the dam.

The temporal evolution of the pore pressure for two nodes is represented in Figure 7. These points are located at the base of the two piezometers, K2 and K3, whose data are reported as well. A free drainage condition was assigned to simulate the presence of drains between the clay core and the rockfill material, as shown in Figure 8. The calculated pore water pressures tend to reach values close to those recorded in piezometer K3. However, the numerical predictions underestimate the pore pressure in correspondence of the position of piezometer K2, although the same trend can be appreciated.

To better understand the source of the difference between the predicted and the observed pore pressure at K2, the latter is compared in Figure 7 with the pore pressure at the same depth corresponding to the current hydraulic head of the reservoir. It can be observed that the two almost coincide, suggesting that the loss in the hydraulic head between the reservoir and the section of the nucleus where the pore pressure is taken is negligible.

Notwithstanding this difference, the numerical model is able to respond appropriately to the enforced temporal variations in the boundary conditions, as shown over the two periods of decrease and increase of the water height in the reservoir reported in Figure 7. This means that the assumptions made in terms of hydraulic properties in saturated and unsaturated conditions are satisfactory.

Figure 9 shows the increment of vertical displacements measured in the geodetic points and simulated through the numerical analyses starting from 06/06/1988. The measured displacements indicate that settlement occurred during the impounding of the reservoir, while the results of the numerical analyses show an opposite trend. This difference originates from the adopted constitutive model: in the numerical simulations the saturation process in the dam body caused by the increasing water level inside the reservoir, induces an increase in pore water pressures and a consequent decrease of effective stresses, which results in swelling of the material. The result shows that the mechanical response of the dam over first impounding cannot be properly described using basic constitutive models, just formulated by substituting Bishop's stress for the effective stress (Lloret & Alonso 1985). The implementation of a more advanced model able to describe plastic collapse upon wetting is deemed mandatory to model the deformational response. Therefore, benchmarking the numerical results on settlements has been abandoned, and no further results are reported in this contribution.

The FS of the downstream and upstream slopes for the four representative situations indicated in Figure 3, are collected in Table 2. As a result of the new pore pressure distribution (Fig. 10) at the end of the first filling of the reservoir, i.e. 17/04/1990, the factors of safety of the downstream and upstream slope experience a decrease of 20% and 25% compared to the configuration after the construction, respectively. As reported in Table 2, a partial decrease of the water level in the reservoir, i.e. 16/12/1992, only affects the FS of the upstream slope, which increases, while the one of the downstream slope remains almost constant.

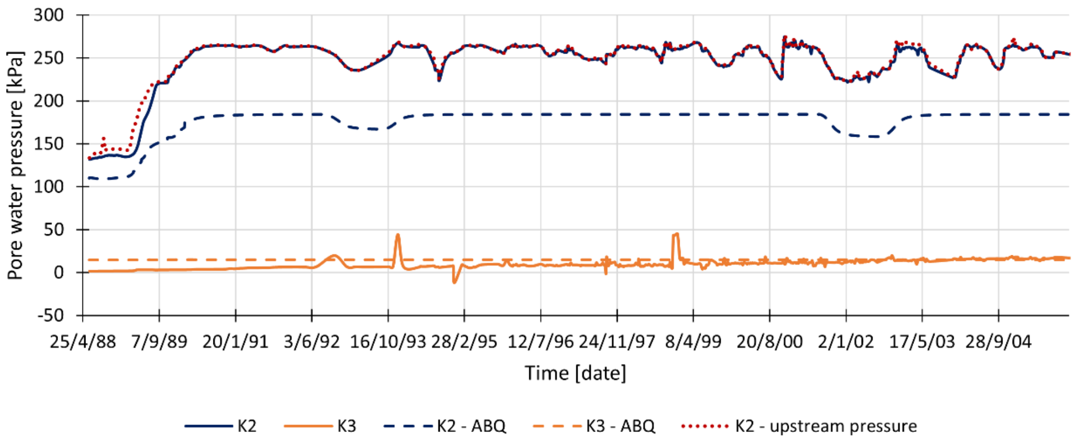

Figure 7. Comparison between pore water pressures measured at the base of piezometers K2 and K3 (solid lines) and results from the numerical analyses (dashed lines), from 25/04/1988 to 03/01/2006. The dotted red line represents the upstream pressure.

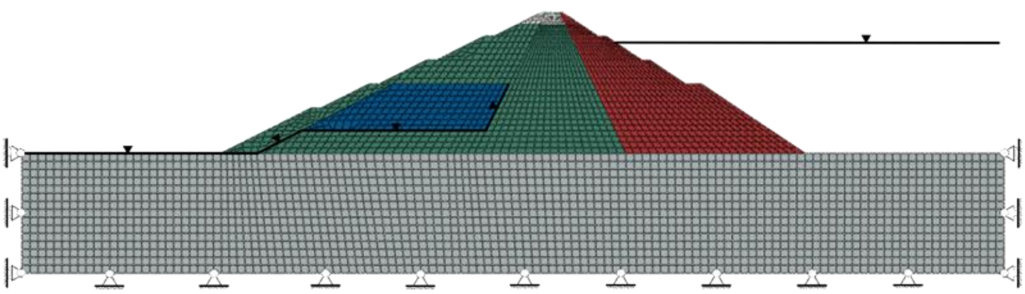

Figure 8. Mechanical and hydraulic conditions applied to simulate Case 1.

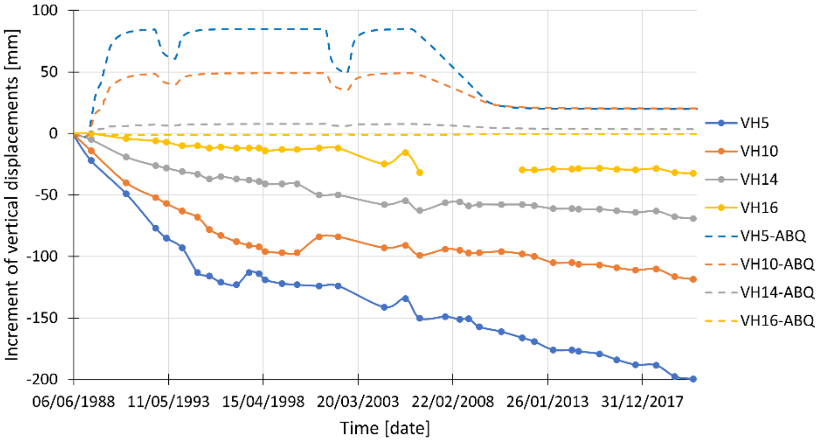

Figure 9. Comparison between the increment of vertical displacements measured in the geodetic points VH5, VH10, VH14 and VH16 (solid lines) and calculated from the numerical analyses (dashed lines) starting from 06/06/1988. Details about the position of the geodetic points are reported in the Formulation document.

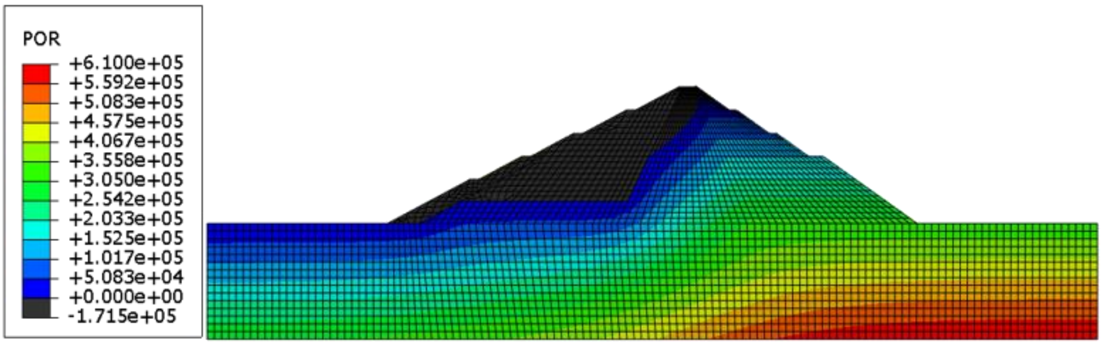

Figure 10. Contour map of pore water pressure inside the dam body and the foundation at the end of the first filling (i.e. 17/04/1990).

Table 2. Factor of Safety for the upstream and downstream slope of the dam for Case 1.

Time	upstream	downstream
As built	1.67	2.20
17/04/1990	1.25	1.76
25/08/1992	/	1.74
16/12/1992	1.48	1.77
13/07/2001	1.25	1.76

3.2 *Case 2 – Task 1: the appearance of the wet stain*

In Case 2 the attention is focused on the appearance of the wet stain, observed on October 24, 2007, at the downstream side of the dam. As already anticipated, the data recorded by the piezometers K2 and K3 (Figure 7), indicate the presence of pore water pressures higher than the ones numerically predicted under ordinary reservoir operation. This mismatch may be originated by localised water losses in the dam body, due to damage experienced by the pipes, well before the appearance of the wet stain. The relevant settlements experienced by the axis of the two pipelines over the construction period (Fig. 11) suggest that damage of the pipes might have originated from the high differential settlements undergone by the dam even before the first impoundment. To evaluate the consequences of this hypothesis, water sources were added at some nodes located along the axis of the pipelines, as shown in Figure 11. After some calibration, the nodes were assigned a specific discharge of 1.e-7 m/s each.

At the same time, the appearance of a wet stain downstream suggested that drainage at the toe should not be constrained. To allow free flow in the dam body without any a-priori restriction on the flow path, the previous downstream drainage was removed. At the face of the downstream slope the non-linear free outflow condition was assumed. The latter allows water to escape from the boundary when the water pressure is above the atmospheric one, while otherwise keeping the boundary impervious. The same condition holds between the core and the rockfill C, in such a way that the drain can be easily modelled as an outflow surface.

The results in the period 25/04/1988 – 30/10/2020 are depicted in Figure 12, in terms of measured and predicted pore water pressures. The fluctuations measured by the piezometer K3 could be related to water management operations, which were disregarded in the numerical simulation. The comparison between the two time-histories confirms that the presence of additional water fluxes from the pipelines could be a comprehensive explanation for the high values of pore water pressures measured in the piezometer K2.

The stability of the system is then evaluated after the wet stain was detected, i.e. 09/05/2008. Results are depicted in Figure 13, for both the downstream and the upstream slopes. For the latter, the increase in the FS (Tab. 3) can be motivated by the decreasing of the pore pressure values with respect to the previous cases.

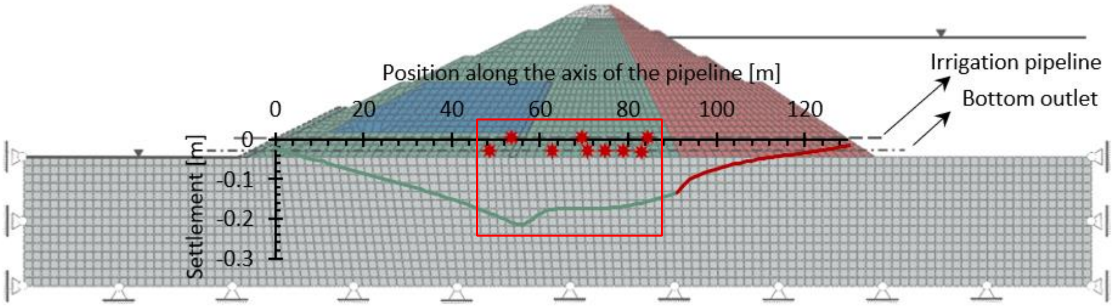

Figure 11. Settlements along the axis of the pipelines at the end of the construction. Different colours represent the different materials. The red marks indicate the nodes selected for the enforced flow.

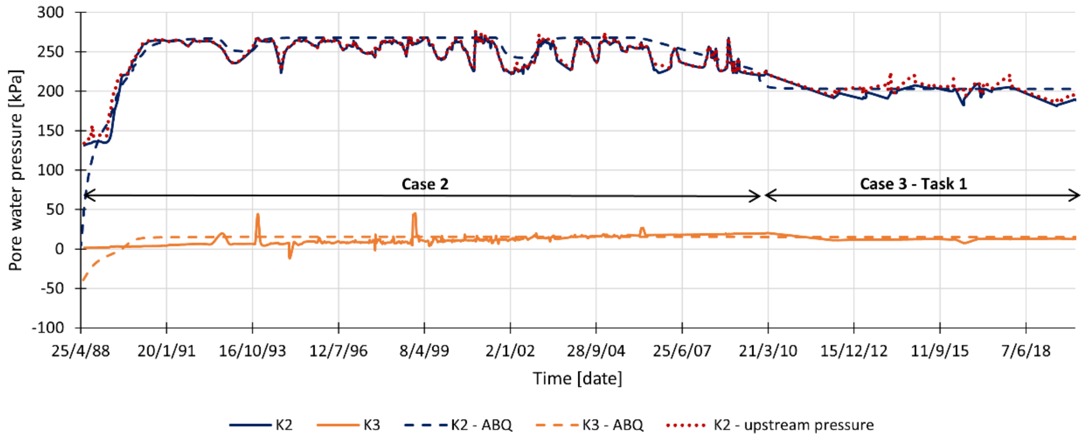

Figure 12. Comparison between pore water pressures measured at the base of piezometers K2 and K3 (solid lines) and results from the numerical analyses (dashed lines), from 25/04/1988 to 30/10/2020. The dotted red line represents the upstream pressure.

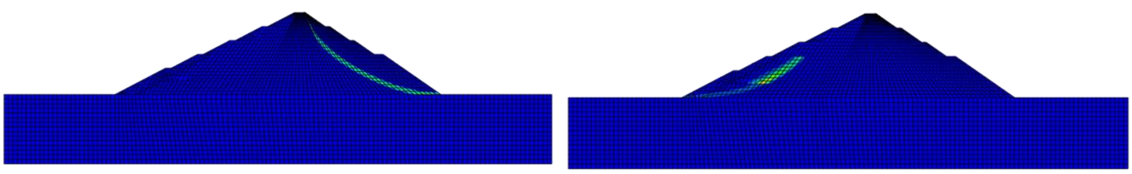

Figure 13. Contour of the plastic deviatoric strain showing the sliding surfaces at upstream (on the left) and downstream (on the right) slope after the detection of the wet stain (i.e. 09/05/2008).

Table 3. Factor of Safety for the upstream and the downstream slope of the dam for Case 2.

Time	upstream	downstream
09/05/2008	2.10	1.75

3.3 *Case 3*

Due to detection of the wet stain, in 2008 the reservoir level was decreased to a maximum level of 93.6 m a.s.l. and the irrigation pipeline was filled with concrete. The water level was lowered again to 92.0 m a.s.l. and the reservoir has operated under such conditions for more than 10 years. In Case 3, the drawdown of the reservoir as well as its re-filling after the remedial works are considered. Attention is devoted also to the effects of the drawdown on the hydraulic properties of the dam core.

3.3.1 *Task 1: the long-term drawdown of the reservoir*

The effect of the long term drawdown of the reservoir can be analysed referring to the data measured between 2010 and 2020, when the reservoir operated at a lowered elevation (Figure 12). Comparing the data measured in the piezometer K2 (solid blue line) with the values corresponding to the upstream pressure (dotted red line), no significant changes in the overall hydraulic response of the clay core have been identified. Therefore, no changes in the material properties have been considered in the definition of the numerical model. It is worth commenting that the difference between these two conditions is slightly higher than the one observed in the previous years (i.e. 1988-2008) since the sealing of the irrigation pipeline carried out in 2008 due to remedial works, led to a reduction of the additional discharge inside the dam body. To reproduce the hydraulic loads experienced on-site by the dam, sealing of the irrigation pipeline has been numerically modelled by removing the water sources along the axis of the irrigation pipeline. The comparison between monitoring data (solid lines) and numerical results (dashed lines) shows that the model is able to reproduce the hydraulic response of the system. At the end of the considered time-window, i.e. 30/10/2020, the stability of the dam is evaluated and the results are collected in Table 4.

3.3.2 *Task 2: filling of the reservoir after the remedial work*

After the completion of the remedial works, the reservoir will be impounded again; based on hydrology data the Formulators estimated that, considering an average rise of 5 cm/day, the normal operating level (i.e. 99 m a.s.l.) will be reached in 1 year (Zvanut et al. 2022). Figure 14 reports the estimated time series of the reservoir level after remedial works.

In the numerical simulations related to the analysis of the future state of the dam, to simulate the permanent sealing of both bottom outlet and irrigation pipelines resulting from remedial works, no additional water fluxes have been assigned in the domain and the seepage of water inside the dam body originates only from impoundment. Since the long-term drawdown of the reservoir could increase the permeability of the material, the hydraulic conductivity of the clay has been increased by one order of magnitude, according to experimental results obtained by Azizi et al. (2020). Particularly, to properly account for the effect of wetting-drying cycles on the hydraulic response of the clay core, only the hydraulic conductivity of the portion of the domain affected by impoundment and drawdown of the reservoir has been modified.

Figure 15 represents the variation in time of pore water pressures at the base of piezometers K2 and K3, as predicted by the numerical model. The pressures inside the clay core (dashed blue line) will be reduced compared to the upstream pressures (dotted red line), suggesting that the remedial works should be effective in reducing the high pore pressures inside the dam previously attributed to damage to the outlet pipelines.

The stability analyses performed at the end of the future filling of the reservoir (i.e. day 337) returns values of FS equal to those obtained in Case 1 (Tab. 4), for time instants characterised by the same hydraulic load (i.e. reservoir water level at 99 m a.s.l.). In fact, the change in the hydraulic conductivity of the material does not significantly affect the pore pressure distribution reached at the end of the re-filling process, as can be observed comparing Figure 10 and Figure 16.

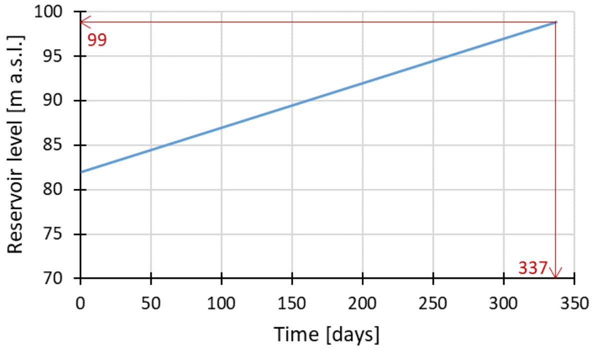

Figure 14. Filling of the reservoir after completion of the remedial works.

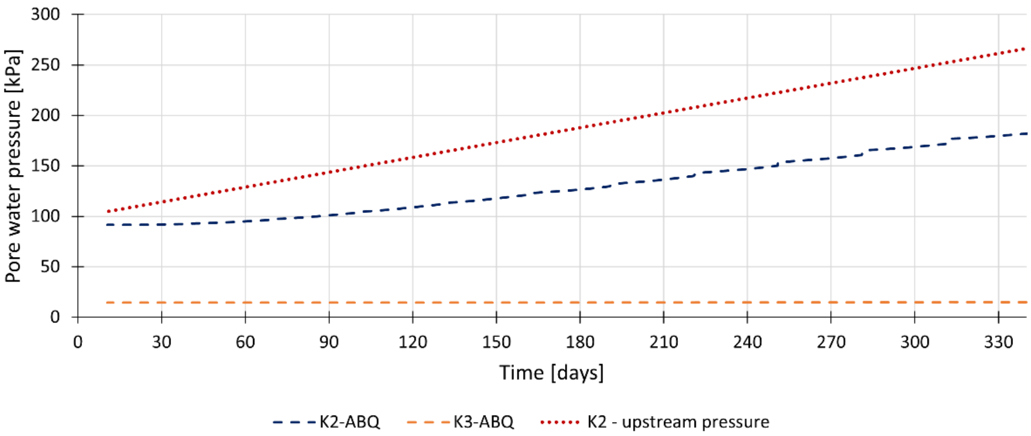

Figure 15. Predicted time-history of the pore pressure field at the base of piezometers K2 and K3 during the new re-filling of the reservoir. The dotted red line represents the upstream pressure.

Table 4. Factor of Safety for the upstream and downstream slope of the dam for Case 3.

Time	upstream	downstream
30/10/2020	1.64	1.75
Day 337	1.25	1.77

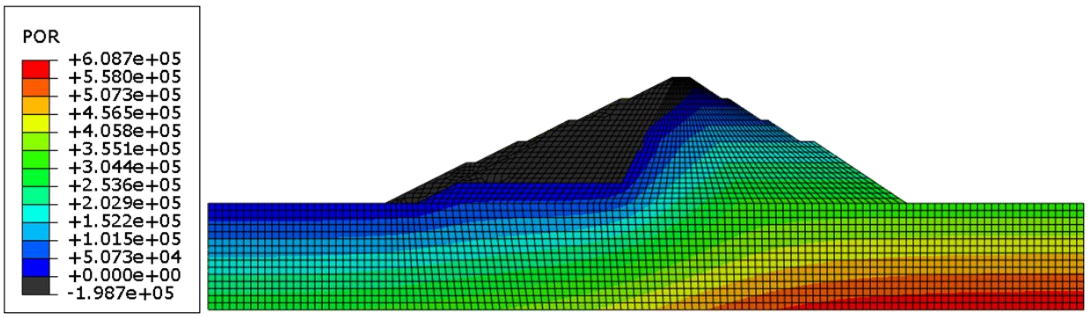

Figure 16. Contour map of pore water pressure inside the dam body and the foundation, at the end of the future filling of the reservoir (i.e. day 337).

4 CONCLUSIONS

In this work the Finite Element Method is used to describe the behaviour of the zoned earth dam presented in the context of the proposed benchmark. Ordinary elastic perfectly plastic constitutive models in a 2D transient consolidation analyses under conditions of partial saturation prove to be able to describe some aspects of the problem, namely the seepage regime and the ultimate failure condition. However, an advanced mechanical modelling would be required to capture the complex time dependent stress-strain response in unsaturated conditions. In particular, the modelling of the material response under wetting and drying paths at constant total stress should be improved. The availability of field measurements must be emphasised, as they provide an essential contribution to the definition of the numerical model. Field measurements are essential not only in the calibration of hydraulic and mechanical properties of the materials, but also in the detection of an anomalous response of the system. The measurement of pore water pressure values higher than expected led to the conclusion that leaks have been taken place inside the dam body for years. The consequent additional water volumes may have led to a significant change in the pore water pressure distribution, resulting in a variation of the effective stress state. Consequently, the stability of the dam is directly affected. The reduction of the safety factor calculated for the downstream slope could be evidence of this effect. Furthermore, this is why the emergency measures put in place, such as the sealing of the original pipes, appear worthwhile.

Additional improvements in the numerical modelling could include the use of a 3D domain, to enhance the contribution of the pipes inside the dam body. Finally, concentrated water flow along the interfaces of heterogeneous materials should be addressed, as it could result in internal erosion phenomena.

REFERENCES

Alonso, E.E., Olivella, S. & Pinyol, N.M. 2005. A review of Beliche Dam. *Géotechnique* 55(4): 267-285

Azizi, A., Musso, G. & Jommi, C. 2020. Effects of repeated hydraulic loads on microstructure and hydraulic behaviour of a compacted clayey silt. *Canadian Geotechnical Journal* 57: 100-114

Caruso, M. & Jommi, C. 2005. An evaluation of indirect methods for the estimation of hydraulic properties of unsaturated soils. *Eastern Mediterraneas University Press, Proc. Int. Conf. Problematic Soils, Famagusta, 25-27 May*

Dassault Systèmes 2021. Abaqus 6.23 Software Documentation. Dassault Systèmes Simulia Corp., Providence, RI, USA

Griffiths, D.V. & Lane, P.A. 1999. Slope stability analysis by finite elements. *Géotechnique* 49(3): 387-403

ICOLD Committee on Embankment Dams 2017. *ICOLD Bulletin 164 Internal erosion of existing dams, levees and dikes, and their foundations*. Paris

Lloret, A. & Alonso, E.E. 1985. State surfaces for partially saturated soils. *Proc. 11th Int. Conf. Soil Mech. Fdn Engng, San Francisco*

Matsui, T. & San, K.C. 1992. Finite element slope stability analysis by shear strength reduction technique. *Soils and Foundations* 32(1): 59-70

Rossignoli, C. & Sterpi, D. 2021. Numerical prediction of the drying-wetting process in a river levee and floodplain. *Proc. 6th World Congress CSEE-21, Lisbon,* DOI: 10.11159/icgre21.lx.111

Wrachien, D. & Mambretti, S. 2009. *Dam-break problems, solutions and case studies*. Boston: WITpress

Zienkiewicz, O.C., Xie, Y.M., Schrefler, B.A., Ledesma, A. & Biĉaniĉ, N. 1990 Static and dynamic behaviour of soils: a rational approach to quantitative solutions. II. Semi-saturated problems. *Proc. Of the Royal Society of London. A Mathematical and Physical Sciences* 1877(429): 311-321

Zvanunt, P., Likar, B., Likar, Z., Selan, V., Klun, M. 2022. Theme C: Behaviour of the embankment dam, Formulation document. ICOLD 16th International Benchmark Workshop on numerical analysis of dams, 5-6 April 2022, Ljubljana, Slovenia.