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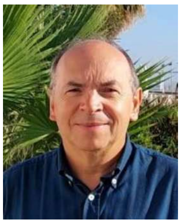
ASSESSMENT OF CONCRETE DAM FOUNDATION DISCHARGES FOR DIFFERENT RESERVOIR LEVELS FOLLOWING A HYDROMECHANICAL DISCRETE MODEL





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ABSTRACT

The ability to anticipate and avoid failures in concrete dams necessarily includes a detailed analysis of the mechanical/hydraulic properties and monitoring data relating to the dam-foundation system, an in-depth analysis of the geometry of the foundation rock mass, complex numerical analysis of dam behaviour and quantitative interpretation models. This ability to assess the safety of the dam-foundation system in an integrated way still needs to be improved, namely by incorporating coupled models that consider the significant interdependence between the mechanical and hydraulic behaviour. In addition, the models for interpreting hydraulic quantities are based on methodologies developed for the interpretation of the mechanical model, which, as a rule, do not fit the observed behaviour and do not consider the coupled behaviour. Coupled hydromechanical analysis of the water flow through the foundation rock mass of a concrete dam is carried using a 2D numerical model that allows seepage through the block interfaces and considers the influence of both grout and drainage curtains. Numerical predictions obtained during cycles of filling and emptying the reservoir are used to establish the influence lines of the hydrostatic pressure on recorded discharges. Two different dam heights and two different foundation geometries are evaluated. Also, two different behaviour scenarios are assessed: i) linear elastic and ii) non-linear behaviour at the concrete/rock and rock/rock interfaces. A series of different polynomial curves are adjusted to the numerical predictions using the least square method to find the best fit equation which can later be adopted in the development of more accurate quantitative interpretation methodologies of recorded data.

Keywords: concrete dams, rock foundations, hydromechanical behaviour, drain discharges.

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1. INTRODUCTION

As mentioned by Enzell et al. [1] dam failure is a complex and highly dynamic process influenced by numerous factors, such as irregular geometries, nonlinear material behaviour, dynamic effects of water release, dam structural movement, and erosion of rock and soil. Anticipating and preventing failures in concrete dams requires a comprehensive approach that includes detailed analysis of the mechanical and hydraulic properties, along with monitoring data related to the dam-foundation system. It also involves an in-depth study of the foundation rock mass geometry, complex numerical modelling of dam behaviour, and the development of quantitative interpretation models.

Monitoring plays a vital role in understanding dam behaviour and preventing failures. Monitoring systems are usually fitted with alarm thresholds that allow the dam owner to detect issues early and take necessary corrective measures. Foundation monitoring results include foundation movement, water pressures and seepage. It is not easy to study the hydromechanical behaviour of dam foundations as this behaviour is non-linear, due to the influence of stress on permeability and to the possible time gap between the variation in the main loads and the variation in the measurement data [2].

The complex coupled hydromechanical behaviour of the foundation rock masses, which has a great influence on foundation seepage behaviour was the subject of several studies in the late 60s and early 70s of the last century [3-5], but still needs to be further studied in situ and adequately analysed using numerical models [6]. In addition, the bond mechanism and the crack development under hydrostatic pressure at the dam/foundation interface add complexity to the dam foundation behaviour and may have an influence on the durability of a dam and/or modify the failure mechanism of a concrete dam on a rock foundation [7, 8].

Regarding modelling of the complex hydromechanical behaviour of fractured dam foundations, equivalent continuum models can be used [9], but more advanced analysis is carried out with fracture flow models in which the discontinuities are explicitly represented [10-12]

In this study, fully coupled hydromechanical analysis of the water flow through the foundation rock mass of concrete gravity dams is carried using a 2D numerical model that allows seepage through the block interfaces and considers the influence of both grout and drainage curtains [11, 12]. Two hypothetical gravity dams of two different heights (15 m and 30 m) are considered. The analysis was conducted under two different behaviour scenarios: i) linear elastic and ii) non-linear behaviour at the concrete/rock and rock/rock interfaces. Numerical predictions obtained during cycles of filling and emptying the reservoir are used to establish the influence lines of the hydrostatic pressure on recorded discharges.

2. HYDROMECHANICAL DISCRETE MODEL

2.1. Fluid flow analysis with Parmac2D-Fflow

The hydromechanical model used in this study, Parmac2D-Fflow, is part of the Parmac2D computational framework [13] and enables the study of the interaction between hydraulic and mechanical behaviours in a fully coupled manner.

The model utilizes interface elements between the mechanical domain blocks and seepage channels (SC) in the hydraulic domain, with these channels aligned along the midplane of the interface elements. The interaction between the blocks, which represent both the rock mass foundation and the dam, occurs always edge to edge. The apertures of foundation discontinuities and water pressures are updated at each timestep, as outlined in [11] and [12]. The rock blocks are assumed to be impervious, with water flow occurring exclusively through the interconnected discontinuities. Water pressures are assigned to the hydraulic nodes (HN), which coincide with the mechanical nodes, and flow rates are computed within the SC.

Flow is modelled using the parallel plate model [4], with the flow rate per unit width of the model expressed by the cubic law. The flow rate [(m³/s)·m⁻¹] in each seepage channel is given by:

$$Q_{SC}=\frac{1}{12\,\mu}\,a_{h.SC}^{3}\,\rho_w\,g\,\frac{\Delta H_{SC}}{L}=k_{SC}\rho_w\,g\,\Delta H_{SC} \quad (1)$$

where 1/(12 μ) = theoretical value of a joint permeability factor (also called joint permeability constant), being μ the dynamic viscosity of the fluid; a = hydraulic aperture of the seepage channel; ρ_w = water density; g = acceleration of gravity; and ΔH = difference in piezometric head between both ends of the discontinuity; and L = length of the SC.

The hydraulic aperture to be used in Equation 1 is given by:

$$a_{h.SC}=a_{h.SC}+\Delta \mathrm{u}_n \quad (2)$$

where $a_{h.SC}$ = hydraulic aperture at nominal zero normal stress and $\Delta \mathrm{u}_n$ = joint normal displacement taken as positive in opening. A maximum aperture, a_{max}, is assumed, and a minimum value, a_{min}, below which mechanical closure does not affect the contact permeability (Figure 1).

Each model block is subdivided into a mesh of linear triangular elements to account for its deformability. The flow is influenced by the stress state within the foundation.

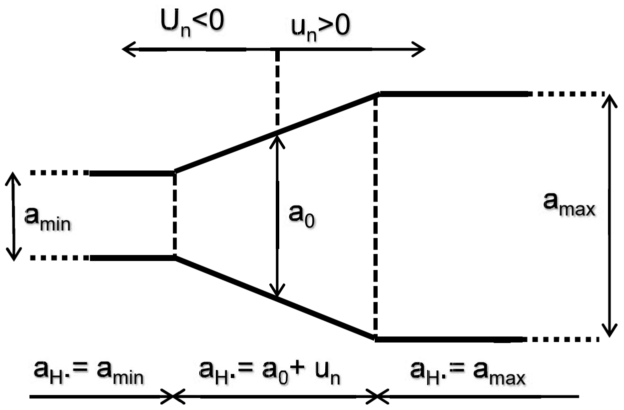


Fig. 1 – Hydraulic aperture

2.2. Model geometry

Two gravity dams of different heights, (15 m and 30 m) are assessed adopting two foundation fracture geometries. Figure 2 shows the schematically geometry of the models and Table 1 shows all the required geometric parameters.

The foundation rock masses were represented with two families of discontinuities. One model, called "reg", was considered, with a family of continuous horizontal discontinuities with a spacing of 5.0 m and a family formed by vertical discontinuities with an average spacing of 5.0 m and a standard deviation of 1.0 m. Another model was considered, called "dip", with families of continuous orthogonal discontinuities that make angles of 15º and 105º with the horizontal. In the latter case, the orientation, and the fact that the families of discontinuities are continuous and flat favours the formation of failure mechanisms, with the formation of wedges that can slide under the dam. In concrete, a set of horizontal continuous discontinuities located 3.0 m apart in the 15 m dam height model and 6.0 m apart in the 30 m dam height model were assumed to simulate dam lift joints.

Table 2 shows the number of mechanical and hydraulic elements used in the four different models. In the gravity dam 15 m high models, Dam 15-reg and Dam 15-dip, the adopted average size of the edges of the triangular elements is 0.5 m, whereas in the gravity dam 30 m high models, Dam 30-reg and Dam 30-dip, the adopted average size of the edges of the triangular elements is 1.0 m.

Figure 3 shows the hydromechanical discontinuum models that were developed for the dam 15 m high, Dam 15-reg and Dam 15-dip. In Figure 3 c) and d) each block is discretized using triangular plane finite elements and interact with others through joint elements. Regarding the hydraulic model, Figure 3 e) and f), the hydraulic head is assumed on the rock mass surface upstream and downstream from the dam (light blue dots); at the drainage system (dark blue dots); at the hydraulic channels (red dots). Impervious lateral and bottom model boundaries are represented by black dots.

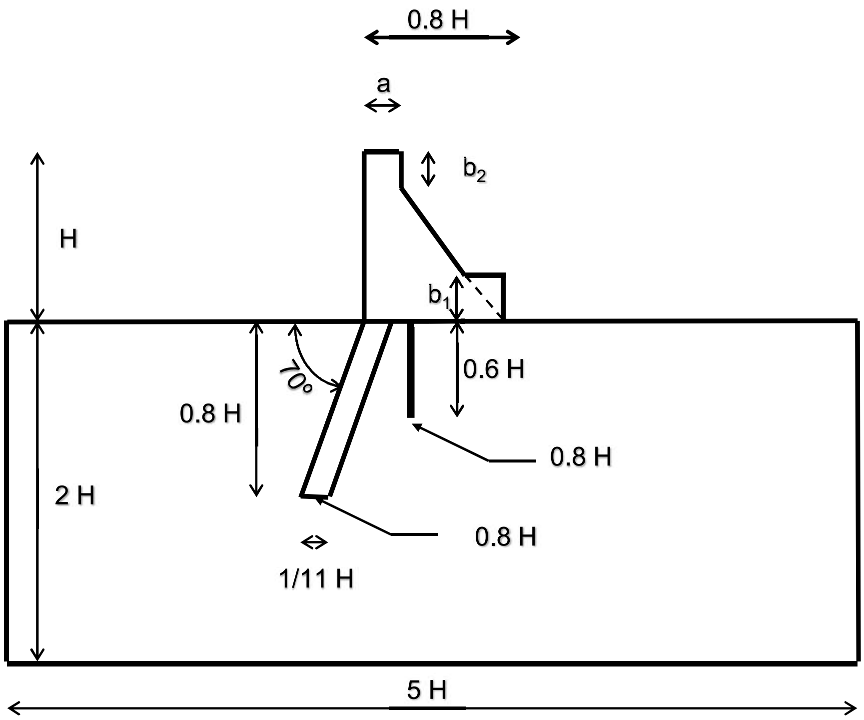


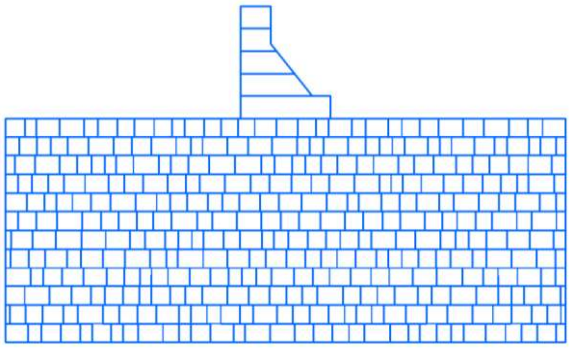
Fig. 2 – Model geometry

Table 1 – Geometry model parameters

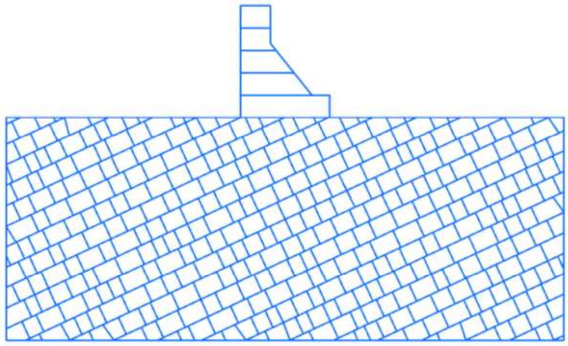
Dam model	H (m)	a (m)	b_1 (m)	b_2 (m)
Dam 15	15	4.0	3.0	5.0
Dam 30	30	7.5	6.0	9.0

Table 2 – Information about the hydro-mechanical discontinuum models

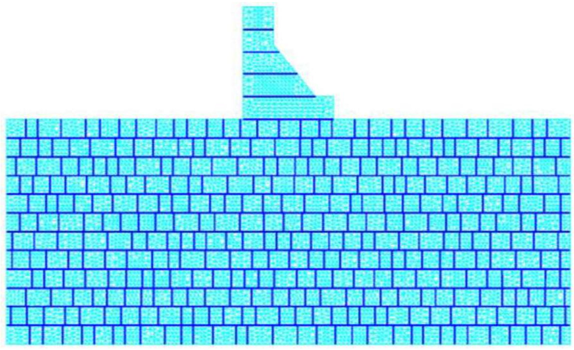
Dam height (m)	Foundation type	Mechanical model				Hydraulic model	
		Blocks	FE triangular elements	FE joint elements	Nodes	Seepage channels	Hydraulic nodes
15	reg	375	15395	3503	11803	3442	3151
15	dip	403	15052	3632	11788	3568	3249
30	reg	387	15425	3659	11974	3446	3155
30	dip	417	15174	3761	11981	3549	3239



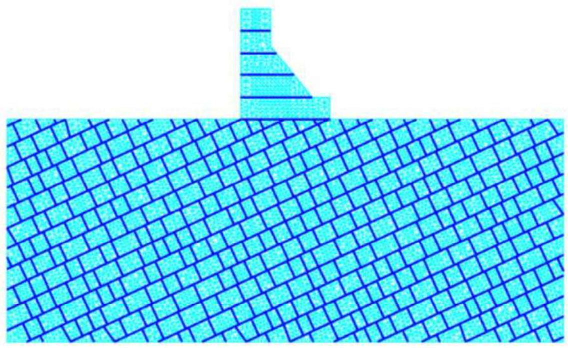
a) Dam 15-reg



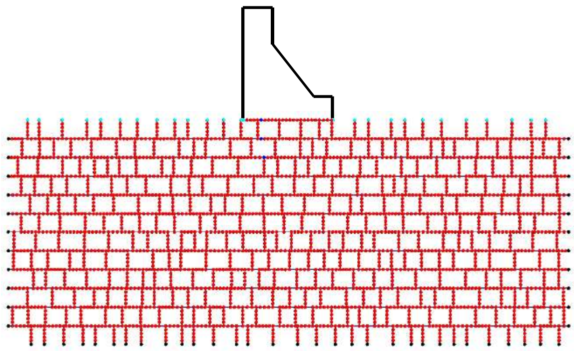
b) Dam 15-dip



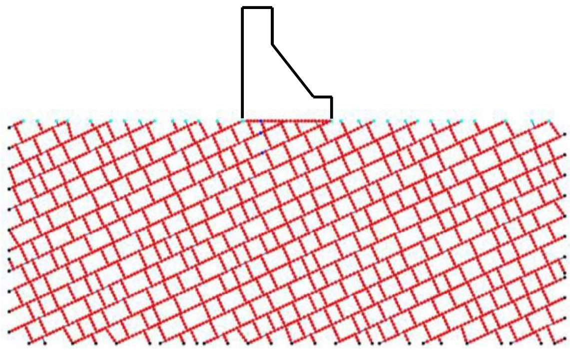
c) Dam 15-reg



d) Dam 15-dip



e) Dam 15-reg



f) Dam 15-dip

Fig. 3 – Hydromechanical discontinuum model of the dam/foundation system of Dam 15-reg (a) and Dam 15-dip (b). Dam and foundation discrete blocks mechanical model ((c) and (d)). Hydraulic model ((e) and (f)). Hydraulic head assumed on the rock mass surface upstream and downstream from the dam (light blue dots); hydraulic head assumed at the drainage system (dark blue dots); hydraulic channels (red dots); impervious lateral and bottom model boundaries (black dots)

2.3. Mechanical and hydraulic parameters

Both dam concrete and rock mass blocks are assumed to follow elastic linear behaviour, with the properties shown in Table 3. A ratio of 0.5 is assumed between normal and shear stiffnesses. Table 3b) also shows the interface contact strength model parameters adopted for the concrete/concrete and concrete/rock interfaces that follow a bilinear softening model. Also shown are the rock-rock interface properties when a nonlinear foundation model is adopted.

The following hydraulic apertures were considered: a_0 = 0.1668 mm, $a_{res} = \frac{1}{3} a_0$ and $a_{max} = 5\ a_0$. It was assumed that the permeability of the dam/foundation interface was half that of the discontinuities in the foundation. The grout curtain permeability is 2.5 times lower than that of the rock mass. The dam construction joints are impervious.

Table 3 – Mechanical properties

a) Volume FE elements

Material	G (GPa)	ν (-)	ρ (kg/m^3)
Dam concrete	30.3	0.24	2.4
Rock mass	64.4	0.2	2.7

b) Joint FE elements

Interface	k_n (GPa/m)	k_s (GPa/m)	σ_t (MPa)	C (MPa)	μ (-)	G_I (Nm/m^2)	G_{II} (Nm/m^2)
Concrete/Concrete	60.6	24.2	2.88	5.76	1.0	87.0	435.0
Concrete/Rock	120.8	51.5	1.37	2.74	1.0	24.7	123.3
Rock/Rock[1]	120.8	51.5	0.0	0.0	1.0	-	-

[1] In the rock mass discontinuities, when non-linear behaviour is considered, zero cohesion and traction and a friction coefficient of 1.0 are assumed.

2.4. Boundary conditions

For the mechanical models, both horizontal and vertical displacements at the base of the models, as well as horizontal displacements perpendicular to the lateral boundaries, were restricted. A pressure corresponding to the hydrostatic pressure was applied at the bottom of the reservoir and on the upstream face of the dam in each model.

Regarding the hydraulic boundary conditions, the dam body and the lateral and bottom boundaries of the model were considered impermeable. It was assumed that there was no water downstream of the dam, and the reservoir was at the same level as the crest of the dam, resulting in a constant pressure at the bottom of the reservoir corresponding to the water height. The drainage system was modelled by applying a water pressure along the drain axis equal to one-third of the hydraulic head upstream of the dam.

2.5. Analysis sequence

The mechanical effect of gravity loads with the reservoir empty is initially applied to the model. Following a hydromechanical analysis is carried out applying successive increments of water head at the reservoir bottom and of hydrostatic pressure at the upstream face of the dam to simulate the rising of water in the reservoir, followed by lowering the water level until the reservoir is emptied.

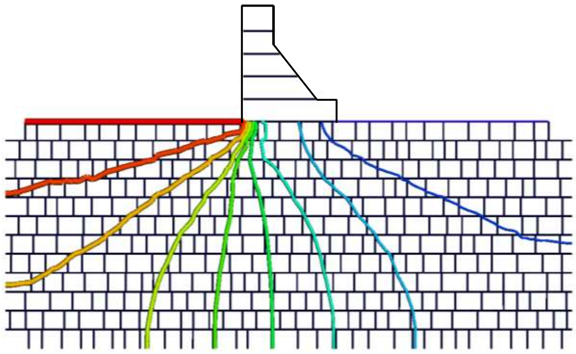
For the two different dam heights and the two different foundation geometries, two different foundation behaviour scenarios are assessed regarding the rock/rock interfaces: i) linear elastic and ii) non-linear behaviour. At the dam foundation interface a non-liner behaviour is always assumed. For the 15 m high dam models a water increment of 2.5 m is adopted whereas for the 30 m high models a 5.0 m water level increment is adopted.

3. RESULTS ANALYSIS

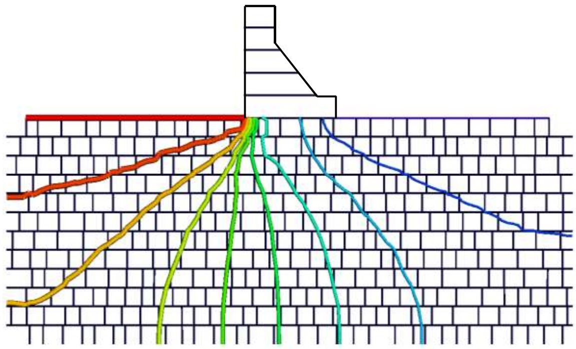
3.1. Fluid flow analysis

Results of the hydromechanical fluid flow analysis, with the reservoir at the same level as the crest of the dam are shown in Figures 4 and 5 for each adopted dam height. The pseudo-equipotentials of piezometric head in the foundation of each dam are presented, in Figures 4 and 5, using percentages of hydraulic head contours within the dam foundation that represent the ratio of the water head measured at a given level, expressed in metres of height of water, to the height of water in the reservoir above that level. Note that, the term pseudo-equipotentials [10] is used due to the discrete nature of the flow, which takes place along the rock mass foundation discontinuities.

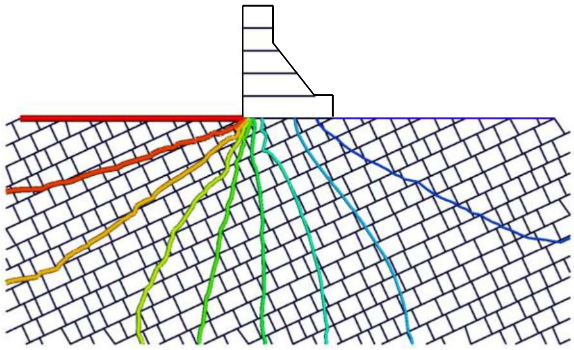
Both figures show that the loss of hydraulic head is concentrated below the heel of the dam, at the drainage and grout curtain’s area. It is also shown that when a nonlinear model is adopted for the rock foundation mass, the contour that represents a 77.8% water head percentage tends to be located at a higher rock mass depth.



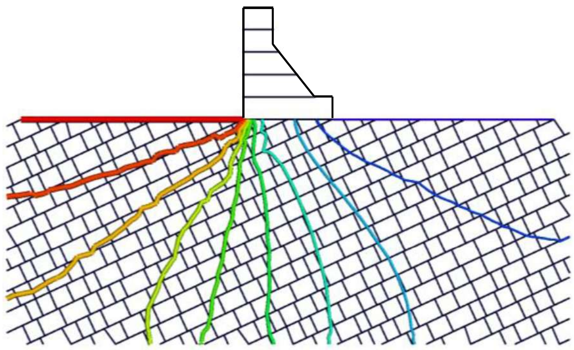
a) Dam 15-reg (EL)



b) Dam 15-reg (NL)



c) Dam 15-dip (EL)



c) Dam 15-dip (NL)

Fig. 4 – Percentage of hydraulic head for full reservoir for the 15 m high dam assuming a rock mass elastic model (EL) and a nonlinear rock mass model (NL)

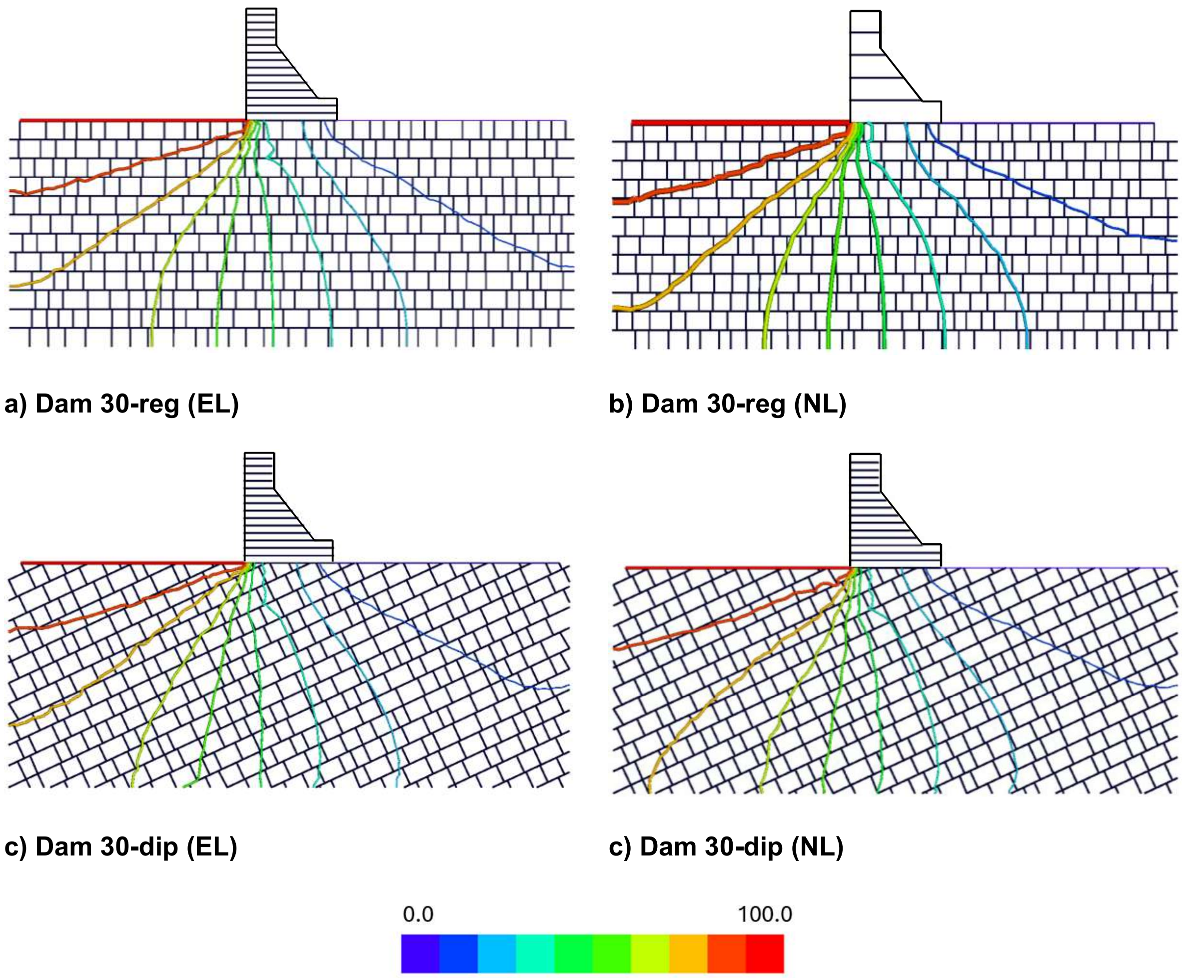


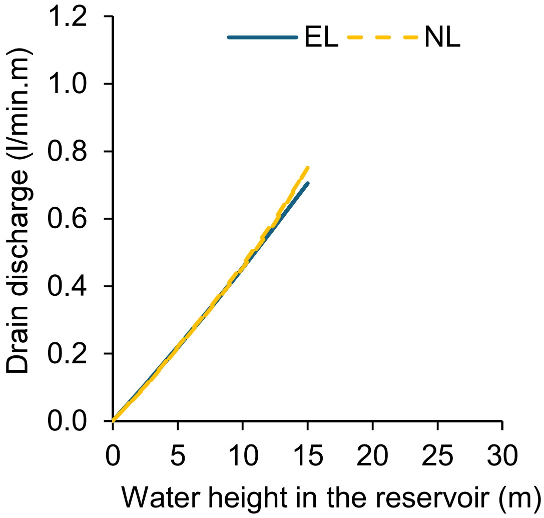
Fig. 5 – Percentage of hydraulic head for full reservoir for the 30 m high dam assuming a rock mass elastic model (EL) and a nonlinear rock mass model (NL)

3.2. Variations in discharges due to variations in reservoir level

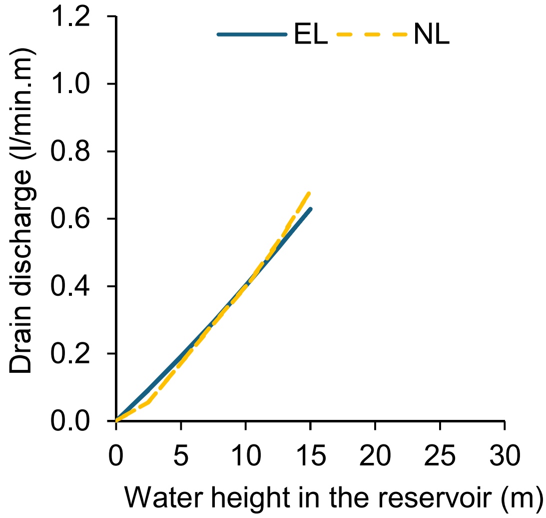
Due to the hydromechanical behaviour of rock masses, the permeability depends on the state of stress and strain within the foundation and, consequently, discharges vary with variations in the reservoir level and resulting changes in rock mass permeability.

For the two different dams, for the two dam geometries and for the two adopted foundation behaviours, a load path, carried out applying successive increments of water head at the reservoir bottom and of hydrostatic pressure at the upstream face of the dam, was followed by an unload path, simulating the filling and a subsequent emptying of the reservoir.

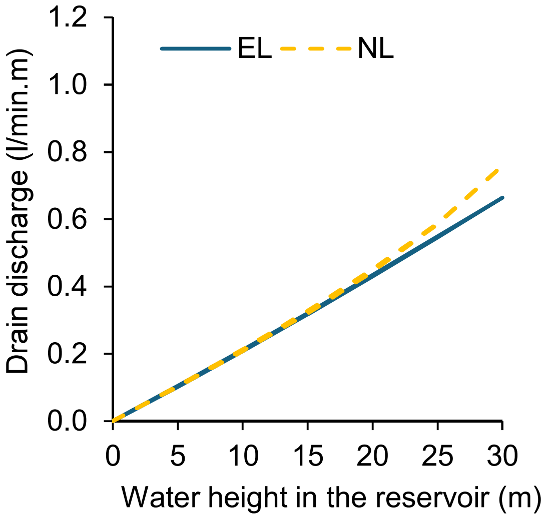
Figure 6 shows the drain discharges for the studied examples. For the dam 30 m high and for the nonlinear foundation behaviour the loading and unloading paths do not match which is due to cracking that occurs at the concrete/rock foundation as the water level rises. This behaviour is more noticeable for the Dam 30-dip model.



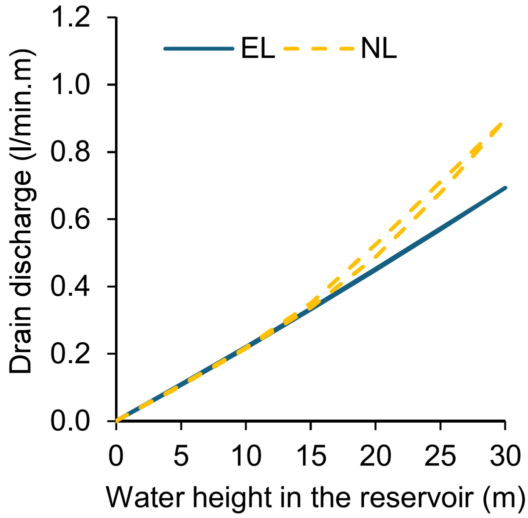
a) Dam 15-reg



b) Dam 15-dip



c) Dam 30-reg



d) Dam 30-dip

Fig. 6 – Calculated drain discharges at different reservoir levels for the two different dam heights, two different foundation geometries and for the two different foundation behaviour scenarios

3.3. Adjustment of polynomial curves

A series of different polynomial curves was adjusted to those shown in Figure 6, regarding the dam 30 m high, using the least square method, to define the functions that better fit the numerical results. Eight polynomial functions were adjusted to the calculated drain discharges. Table 4 shows the different polynomial functions and the correlation coefficients that were obtained, where the best fits are highlighted in bold.

Table 4 – Correlation coefficients of the prediction functions

Polynomial function	Model			
	30-reg (EL)	30-reg (NL)	30-dip (EL)	30-dip (NL)
h^4	0.303085	0.467299	0.305463	0.580904
h^3	0.558291	0.682296	0.560148	0.767186
h^2	0.835983	0.901082	0.837044	0.924246
$h^4 + h$	0.999908	**0.999985**	0.999902	0.999484
$h^3 + h$	0.999963	0.999873	0.99996	**0.999922**
$h^2 + h$	**0.999999**	0.999203	**0.999999**	0.999093
$h^4 + h^2$	0.961773	0.968812	0.962037	0.98035
$h^3 + h^2$	0.975402	0.978243	0.975577	0.98571

Table 4 analysis leads to the conclusion that a function of the type q = f (h^2, h) would accurately fit the numerical results for both the 30-reg and 30-dip models, assuming linear elastic behaviour of the dam foundation. Assuming a non-linear behaviour both at the concrete/rock and rock/rock interfaces, a function of the type q = f (h^4, h) would better fit the numerical results for the 30-reg foundation, and a function of the type q = f (h^3, h) would better fit the numerical results for the 30-dip foundation.

Equations 3 to 6 are the functions that better fit the numerical results, for the four different models, respectively, 30-reg (EL), 30-reg (NL), 30-dip (NL) and 30-dip (NL):

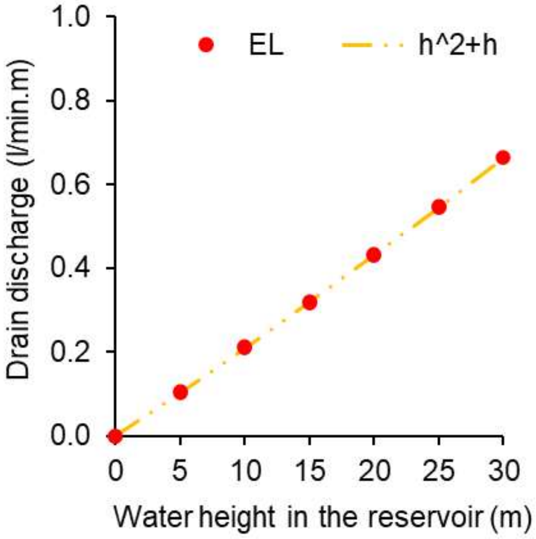
$$q = 6.40\text{x}10^{-5}\, h^2 + 0.020215\, h \tag{3}$$

$$q = 1.68\text{x}10^{-4}\, h^4 + 0.020784\, h \tag{4}$$

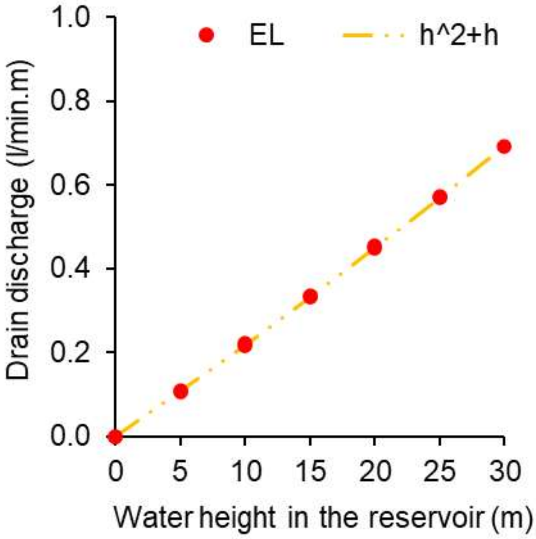
$$q = 6.91\text{x}10^{-5}\, h^2 + 0.021048\, h \tag{5}$$

$$q = 1.07\text{x}10^{-5}\, h^3 + 0.020274\, h \tag{6}$$

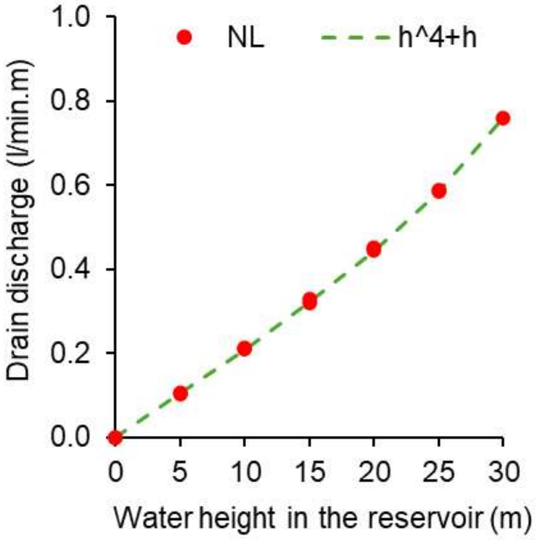
The comparison between numerical results and those obtained with the adjusted polynomial curves that better fit the results is presented in Figure 7. Figure analysis clearly shows that the graphs of the adjusted curves follow adequately the curves which represent the variation in discharges due to variations in reservoir level.



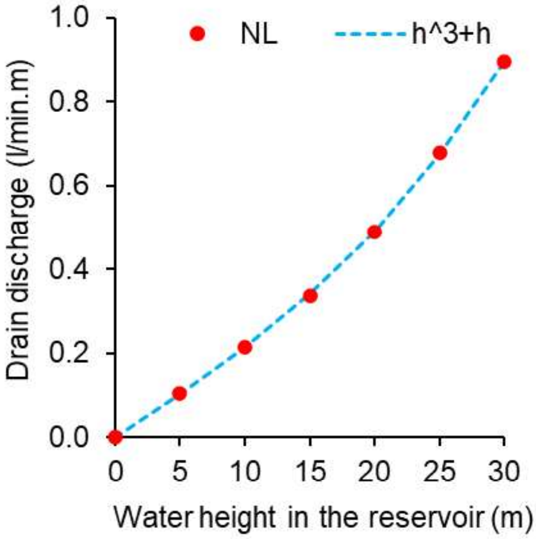
a) Dam 30-reg (EL)



b) Dam 30-dip (EL)



c) Dam 30-reg (NL)



d) Dam 30-dip (NL)

Fig. 7 – Calculated drain discharges at different reservoir levels for the 30 m dam height, for the different foundation behaviours, and functions that better fit the numerical results

It must be highlighted that, in practice, the comparison of recorded discharges with those calculated using such functions may allow anomalous foundation behaviour to be identified.

4. CONCLUSIONS

This paper presents a study on seepage through gravity dam foundations using discontinuum models. The analysis focused on two hypothetical gravity dams of two different heights (15 m and 30 m), and, for each height, two different foundation geometries. A two-dimensional model, Parmac2D Flow, was used, which considers the coupled hydromechanical behaviour of rock masses. For each dam, the study examined the distribution of hydraulic head within the foundation and the amount of water flowing through the models under different scenarios involving foundation discontinuity pattern and foundation behavioural assumptions.

The results demonstrate the significant impact of the grout curtain and drainage system on the distribution of water pressures. The volume of water flowing through the dam foundations was found to be greater, for the highest reservoir levels, when a non-linear behaviour is assumed, compared to the results from linear elastic analysis.

The study also allowed the determination of the hydrostatic pressure influence line on discharges for each dam height, and polynomial functions were developed to accurately represent the shape of these lines.

Ongoing research is focused on studying the influence of the aperture of the foundation discontinuities on numerical discharges and establishing a unified polynomial function that incorporates both the calculated discharge values and the dam height. Similar studies are also going to be carried out using the 3D version of the Parmac2D code (Parmac3D-Fflow), that allows the three-dimensional behaviour of dams to be considered.

ACKNOWLEDGMENTS

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