Abstract

This work details a preliminary design for a gravity dam to be built with roller compacted concrete (RCC), located in central Portugal. Some design aspects are emphasized, related to the adoption of a tentative section with gallery satisfying geometric and construction constraints, as well as aspects associated with safety of the structure and adjoining foundation. Although a preliminary analysis for the initial geometry proved structurally safe, a plane stress finite element analysis indicated the need for geometric refining insuring safety under a specific seismic load combination. An improved section for the dam was achieved, inducing safe detailed patterns of stresses and displacements for the dam and foundation.

Keywords: dams, roller compacted concrete, finite elements, optimised shape, preliminary design, Ribeiradio RCC dam.

1 Introduction

The mankind constant need of water for its survival motivated, throughout the ages, the construction of dams. In particular, the one referred in this work will be located in river Vouga in central Portugal, near Ribeiradio village and 8 km upstream of the town of Sever do Vouga.

With a storage capacity beyond 10^8 m^3, it will constitute a source for public water supply and industrial water supply for both population/industrial demands of townships along Vouga hydrological basin, as well as for agriculture usage in the fertile Vouga valley. Additionally, it will also be quite useful in river flood control of the lower part of the river Vouga, dampening the effects of characteristic flood rates and reducing flood frequency and severity, namely in the vicinity of the city of Águeda. The hydraulic structure will still provide a hydroelectric power of 10 MW, also important for the paper-pulp industries of the zone.

As outlined by Hansen and Reinhardt [1] roller-compacted concrete (RCC) has become one of the most widely used materials for modification and upgrading of aging embankment dams in the USA, with inherent dam safety and appearance. Moreover, the RCC techniques made also possible to use RCC gravity dams as an economically competitive alternative to conventional concrete or embankment dams. The possibility of rapid construction at low cost constitutes the major advantage of RCC dams. Because of their horizontal method of construction, stepped spillways and outlet works are incorporated as an integral part of the RCC structure, with a reduced velocity at the toe of the spillway. Also because of shorter construction periods, the river diversion and the potential need of cofferdams are significantly minimised, especially for large rivers.

Based on these advantages and also on the facts that the crest elevation and footprint for a RCC dam are considerably and invariably less than for embankment dams, it is not surprising that the Portuguese authorities planned a RCC dam for such a convenient site. Moreover the specification of continuous placement of RCC, minimising cold joints between horizontal layers, would allow contractors bids at lower unit prices for a dam construction with little risk.

Tendering and the subsequent bidding are increasingly becoming an essential component of procurement in a wide range of industrial private and public sectors, including the service industry of engineering consultancy and construction. As owner of the future hydraulic structure, Portuguese Water Resources and Environment Ministry (INAG 'Instituto da Água') promoted in June 2000 an international tendering, involving simultaneously conception and construction of the RCC dam, in order to select the most technically qualified construction consortium.

The 6 distinct construction partnerships that answered to the call until December 2000, organized their candidacies, proposed alternative technical solutions and submitted their construction bids, based on a provided reference solution of a preliminary study conducted earlier by COBA in 1999 [2]. This permitted INAG to classify and rank partnerships through more open, transparent, fair, more coordinated, efficient and more cost-effective selection practices.

2 Improved Alternative Solution for Ribeiradio Dam

The preliminary design calculations and subsequent finite element analysis, partially shown in the present paper, resulted from consultancy services permitting the attendance of a participating partner in the international public tendering promoted by INAG in 2000, associated with the construction of the above-mentioned RCC dam. It is typical of similar analysis done at standard civil engineering consultancy offices throughout Portugal, as a clear indication of continuous technical updating and upgrading in the art of engineering design.

In order to permit a fair biding of the construction costs of the required complete dam infrastructure, an alternative more economic solution was searched, whose structural strength integrity and safety had to be assessed. The alternative solution proposed was based on minimising the construction costs per unit length, on the basis of the following sketch of fixed and variable parameters of an idealized simplified cross section of the gravity dam (Figure 1).

The restrictions established on these geometric parameters where either operational or on the basis of the reference solution [2]. Such set of optimised dimensions (minimising costs and therefore cross section areas) permitted a preliminary efficient evaluation of dam safety as a beam under composite flexure and of its stability as a rigid body. Only under these circumstances would a further finite element analysis be performed for the evaluation of the generalized stresses and displacements in the dam core, interface and foundation medium.

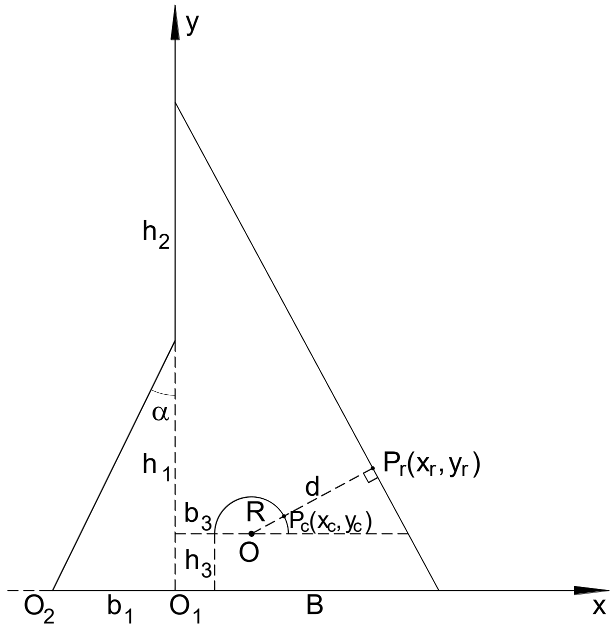


Figure 1: Sketch of the cross section geometry for optimisation

Assume B=51.75 m, H=64.25 m, b_3=7.75 m and h_3= 10.75 m to be fixed quantities as given by the reference solution [2]; also assume b_1, h_1 and R to be variable quantities subjected to the restrictions

$$\begin{cases} H = h_1 + h_2 \\ d = |P_r - P_c| > d_{\lim} \geq 5m \\ b_1 = h_1 \, tg\,\alpha \\ 0.4\,\mathrm{H} < \mathrm{h}_1 < 0.6\,\mathrm{H} \end{cases}$$

such that $x_0 = b_3 + R$ and $y_0 = h_3$.

Minimising the area $A = \frac{BH}{2} - \frac{\pi R^2}{2} + \frac{b_1 h_1}{2} = \frac{BH}{2} - \frac{1}{2}\left(\pi R^2 - b_1 h_1\right) = \frac{BH}{2} - \frac{\Delta A}{2}$ is equivalent to maximising the area increment $\Delta \mathrm{A} = \pi \mathrm{R}^2 - \mathrm{h}_1^2 \, \mathrm{tg}\,\alpha = \pi \mathrm{R}^2 - \mathrm{b}_1\,\mathrm{h}_1$. But more precisely it is equivalent to maximising the difference in costs per unit length, associated with the substitution of RCC from the dam core by RCC and structural concrete at the bottom apron of the upstream face. Such cost difference is safely expressed by $\Delta A_{\cos t} = C_{RCC}\,\pi R^2 - C_{SC}\,b_1\,h_1$, where C_{RCC} is the local unit cost of an m^3 of RCC and C_{SC} is the local unit cost of a m^3 of structural concrete.

Conceptually the potential economic gains would be associated with the creation of a large cavity or gallery at the dam core and its substitution by an inclined bottom area also of RCC but with a surface finishing in structural concrete to be continuously submerged. The construction, economic and technical advantages of using RCC would still be fully exploited, with a very small increase of the costs of the upstream structural concrete plus the costs of slip form or precast concrete gallery units or of conventional forming method. Further geometric deductions detailed by Barros [3] permitted to obtain the following Table 1 of possible radius for the geometric implantation of the semi-circular gallery at the dam core.

d_ℓ (m)	10	11	12	13	14	15	16	17	18	19	20
R_ℓ (m)	9.85	9.3	8.7	8.2	7.6	7	6.5	5.9	5.35	4.8	4.2

Table 1: Limiting radius $R_{\ell im}(d_{\ell im})$ for the geometric implantation of the gallery

Local construction indexes relate approximately RCC and structural concrete costs by $C_{RCC} \approx 60\% \, C_{SC}$, so that the function to be maximised is now $F(R, s_g) = 0.6\pi R^2 - 1032.0156\, s_g$. From this it follows that the limit cavity radius corresponding to null gain (or null economic added value) would be $R = \sqrt{547.5\, s_g}$ where s_g is the slope gradient of the upstream bottom face. For an initial choice of $s_g = 10\%$, permitting to minimise corners and concrete formworks at the bottom upstream face near the water intake, it follows that $R_{\text{limit}} > \sqrt{547.5 \times 0.10} \approx 7.39\, m$ beyond which effective economic gains occur. Using $R = 7.5\, m$ as initial value of the arc radius, then $d_\ell \approx 14\, m$ and $b_{1_\ell} \approx 3.35\, m$.

Evaluation of dam safety as a beam under composite flexure, calculated the compressive stresses at the downstream and upstream feet of the dam core as 1.0 and 0.4 MPa respectively. Since sufficient slack appears to exist for the compressive stress values at dam feet, further economic gains where attempted by extending vertically (about 12 m) the cavity to the foundation, therefore assuming an arc-rectangle shape. The safety of the proposed RCC dam with this new arc-rectangle gallery was then ascertained with respect to its strength, stiffness and base stability, under several combinations of load cases.

The preliminary strength study performed by Barros [3] is synthesized in Table 2, where safe values for the normal stresses (+, compressive) at the upstream and downstream dam feet $(\sigma_u \text{ and } \sigma_d \text{, MPa/m})$ are given for different load cases. These load cases are combinations of self-weight W, hydrostatic pressure P_h without and with bottom-uplift U, and horizontal earthquake E acting on the empty and full reservoir in two possible senses (upstream to downstream $u \rightarrow d$, and inversely). The seismic actions are modelled by equivalent static forces on the basis of the maximum local seismic acceleration $a_{seismic} \approx 0.1g \;\; m/s^2$ (with return period of 1000 years), proposed by Oliveira [4] in the standard characterization of seismic hazards and evaluation of the seismic risk in Portugal.

Load Case	σ_u	σ_d
W	1.20	0.12
W+P_h	0.74	0.66
W+P_h+U	0.10	0.66
Empty reservoir W+$E_{u\to d}$ W+$E_{d\to u}$ W+$E_{d\to u}$ (RCC without tractions)	 1.04 1.36 1.57	 0.28 -0.04 0.00
Full reservoir W+P_h+U+ $E_{d\to u}$ W+P_h+U+ $E_{u\to d}$ W+P_h+U+ $E_{u\to d}$ (RCC without tractions)	 0.34 -0.14 0.00	 0.42 0.90 1.11

Table 2: Load cases for strength study

The stiffness study is performed calculating the dam crest total lateral displacement and comparing it with admissible values. The lateral displacement contributions, estimated according to the practical methodologies proposed by Herzog [5], take into account flexure and shear deformations of the concrete $(w_{conc} = 4.45\,mm)$, flexure and shear deformations of the foundation $(w_{found} = 5.68\,mm)$, upstream rotation of the valley bottom $(w_{rot} = 2.25\,mm)$ and thermal gradient between upstream and downstream faces $(w_{temp} = +/-1.84\,mm)$ measured conveniently 1 m below the surface and inside the dam core concrete mass. The total estimated dam crest lateral displacement varies between a minimum of –1.84 mm ≈ -2 mm upstream (when the reservoir is empty and the downstream face is the warmest) and a maximum of 4.45+5.68-2.25+1.84=9.7 mm ≈ 1 cm downstream (when the reservoir is full and the downstream face is the coolest).

The base stability of the dam (with arc-rectangle gallery) against sliding as a rigid block was evaluated without and with earthquake occurrence, assuming the indentation of the dam core on the foundation base as suggested in the reference solution. The safety factors obtained were respectively 1.5 and 1.2, using very small stress values of 0.1 MPa for the adherence between the concrete and the foundation.

3 Global Stability Analysis

The design and constructed quality of lift surfaces are critical to the stability and seepage performance of the RCC dam. Since RCC is often placed in 250-400 mm thickness layers and subsequently compacted, the global stability of the RCC dam is an essential issue to verify at each dam elevation associated with successive lifts.

The need to minimise the number of cold joints, between horizontal layers of successive lifts, is also indicative of the number of verifications of RCC dam global stability at each elevation, calculating the sliding safety factor according to the following relation between the active and reactive forces:

$$F_{SLIDING} = \frac{\sum \text{Reactive forces}}{\sum \text{Active forces}} \tag{1}$$

The active forces result from the water pressure acting on the upstream face of the dam, while the reactive forces are guaranteed by the horizontal frictional forces FH_{RD} mobilized in the surface between concrete layers of RCC or between concrete and the foundation soil. The latter are expressed by:

$$FH_{RD} = \left(\sigma_n tg\phi_d^{'} + c_d\right) A \tag{2}$$

where σ_n are the local normal stresses at each element area A, c_d is the design cohesion and $\phi_d^{'}$ is the design friction angle. The design values of the geotechnical parameters (cohesion and friction angle) were obtained from the prescriptions of Eurocode 7 [6], dividing the characteristic values c_k and $\phi_k^{'}$ by the corresponding partial safety factors, respectively γ_c and γ_ϕ, as expressed in equations (3) and (4).

$$\phi_d^{'} = \frac{arctg\left(tg\phi_k^{'}\right)}{\gamma_\phi} \tag{3}$$

$$c_d = \frac{c_k}{\gamma_c} \tag{4}$$

The partial safety factors γ_ϕ and γ_c are shown in the following Table 3, for the two types of analyses to be performed.

Analysis	$\gamma_{\phi'}$	γ_c
Static	1.5	4.0
Dynamic	1.1	3.0

Table 3: Geotechnical partial safety factors

The characteristic values of the geotechnical parameters for global stability analysis are given in Table 4, for the two types of interface zones between contacting materials.

Surface	$\phi_k^{'}$ (degrees)	c_k (kPa)
Concrete-concrete (RCC)	42.0	550.0
Concrete-foundation	45.0	400.0

Table 4: Characteristic values of geotechnical parameters

The potential sliding stability at the elevations of the cold joints proposed in the reference solution [2] was ascertained, and the corresponding sliding safety factors are synthesized in the following Table 5 extracted from [7].

Height of Sliding Surface (m)	Sliding Safety Factor
40	1.35
50	1.49
60	1.60
70	1.81
80	2.08

Table 5: Sliding safety factors for different heights

4 Finite Element Analysis

Since the previous preliminary analyses have been successfully performed for the original and improved optimised cross section, ascertaining the overall or global safe performance and behaviour of the RCC dam, a detailed local analysis by the finite element method seems now quite justifiable. Such analysis will characterize the pattern of generalized stresses and displacements of the complete model, namely in the RCC core, at the interface between dam and foundation, and in all the modelled portion of the foundation soil.

4.1 Finite Element Model

The structural behaviour of the dam's critical cross section (Figure 2) was modelled with an eight-node parabolic finite element mesh, in order to simulate the plain stress state installed in the concrete of the RCC dam and in the material of the foundation underneath.

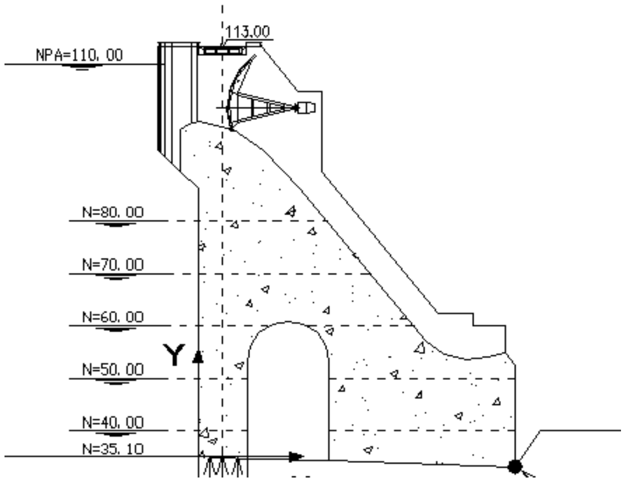
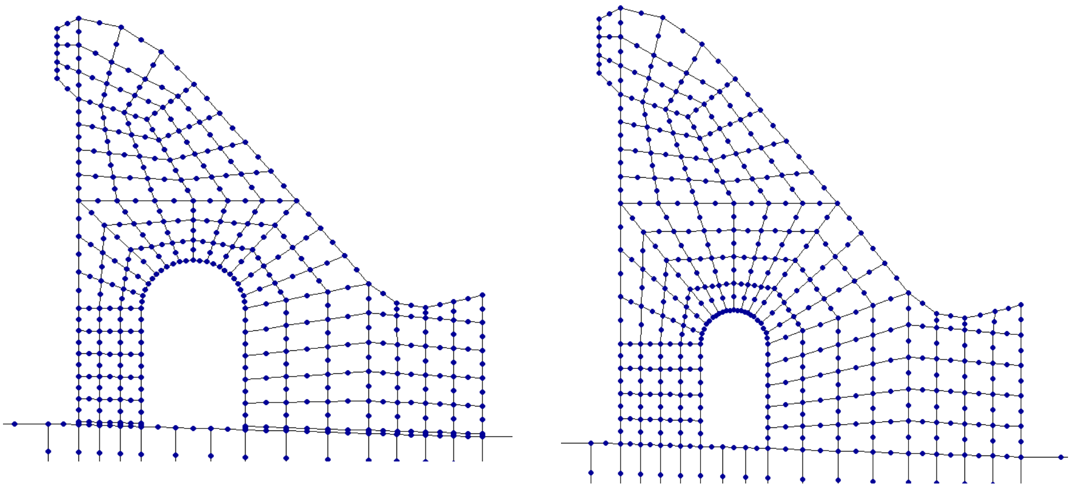


Figure 2: Dam critical cross section

Since on one hand this was a preliminary study and on the other hand a cost minimisation (optimisation of the weight lightening gallery with respect to dimensions and location), a coarse mesh was used to obtain a first approximation of the stress level and the magnitude of expected displacements for each tested geometry.

The two meshes tested during finite element modelling are visualised in Figure 3, obtained through the use of VIFEM software (Visual Interface for Finite Element Method) developed by Teixeira [8]. The original geometry refers to the arc-rectangle gallery proposed for preliminary analysis; the improved geometry corresponds to a decrease of its dimensions with a more downstream implantation in the dam core.



Original geometry   Improved geometry

Figure 3: Finite element meshes

The structural behaviour of the dam could only be modelled with acceptable accuracy taking in account the existing interaction between the RCC core and the foundation material on which the dam rests. According to Figure 4 a certain amount of surrounding soil was also meshed, with the purpose of minimising the boundary effects due to the finite character of the mesh.

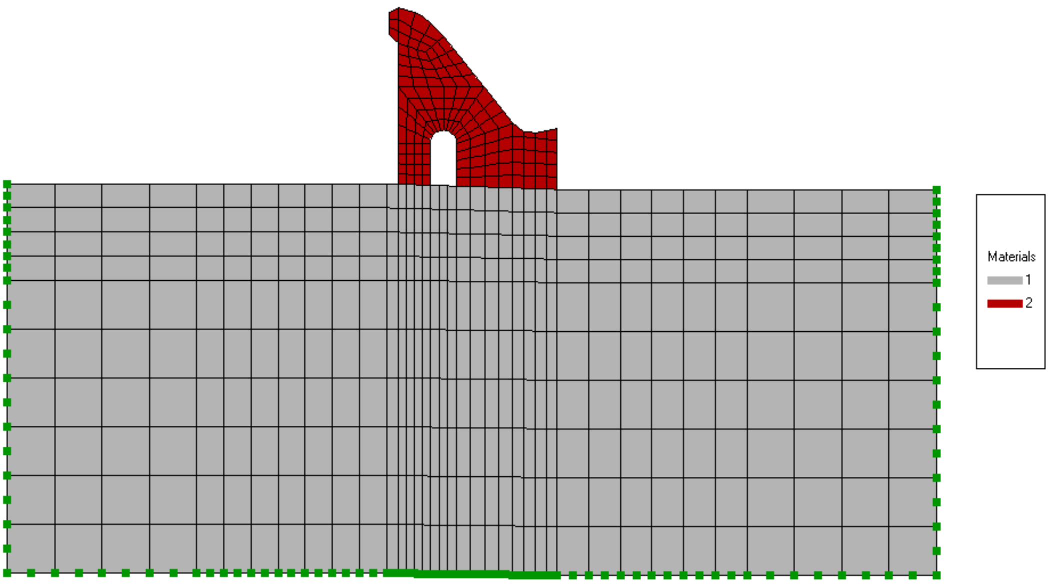


Figure 4: Finite element mesh of dam and foundation, for the improved geometry

The boundary conditions considered correspond to a rigid link to the bottom layer of the foundation material and horizontal restriction to movement on both vertical boundaries of the meshed domain.

The mechanical properties of the materials involved in the simulation of both the foundation and the RCC core of the dam are shown in the following Table 6, where E is the elastic modulus, ν the Poisson ratio and γ the specific volumetric weight.

Material	E (GPa)	ν	γ (kN/m^3)
1 - Foundation	2	0.35	19
2 - RCC	20	0.22	24

Table 6: Material data for the design of the RCC dam

4.2 External Loads

The external loads applied to the RCC dam for this finite element analysis are the sum of two kinds of forces. The first kind corresponds to mass forces f_{mx} and f_{my} due to seismic and gravity accelerations, respectively in the x and y directions, calculated per unit volume by:

$$f_{mx} = \frac{a_{seismic}}{g} \gamma_{concrete} = 2.45\, kN/m^3 \tag{5}$$

$$f_{my} = \gamma_{concrete} = 24\, kN/m^3 \tag{6}$$

The term $a_{seismic}$ is the maximum horizontal ground acceleration (here acting in the body of the dam), which was already justified to assume the value of 1.0 m/s^2.

The second kind of forces represents the external pressures induced by the water on the upstream face of the dam (Figure 5), including the hydrodynamic term accounting for the seismic effect in the water mass, calculated with Westergard's parabola [9]. The hydrostatic and hydrodynamic pressures are given respectively by

$$p_{hydrostatic}(y) = \gamma_{water} (H - y) \tag{7}$$

$$p_{seismic}(y) = K \left(1 - \frac{y^2}{H^2} \right) \tag{8}$$

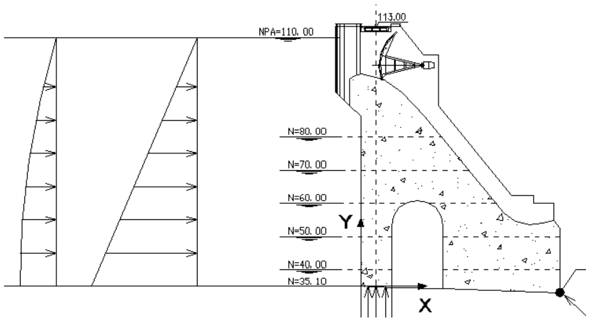


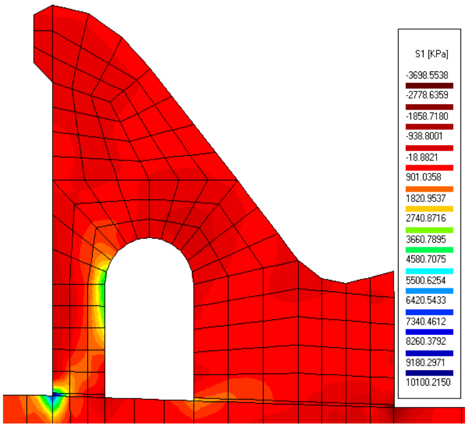
Figure 5: Hydrostatic and hydrodynamic pressures on the upstream face

where $H = 75\,m$ is the maximum height of water in the reservoir, γ_{water} is the water volumetric specific weight and the scaling factor $K = \frac{a_{seismic}}{g}\,\gamma_{water}\,H$.

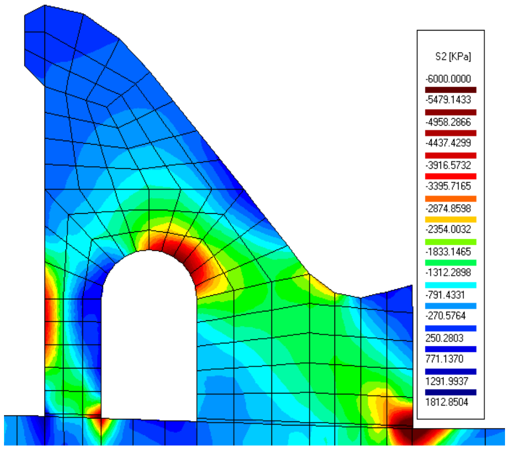
4.3 Stresses and Displacements

In the next figures are shown the results of the stress and displacement finite element calculations associated with the seismic load case, in terms of principal stresses ($\sigma_1,\sigma_2,\sigma_3$) and total displacements of the deformed mesh, also obtained using VIFEM software [8]. Figures 6 to 9 refer to the preliminary original geometry of the arc-rectangle gallery at the RCC dam core.

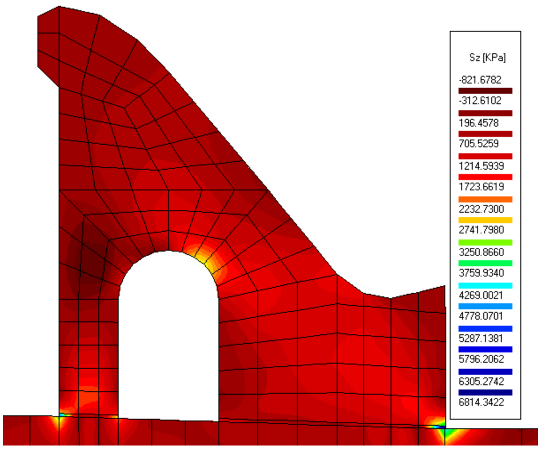
Preliminary original geometry



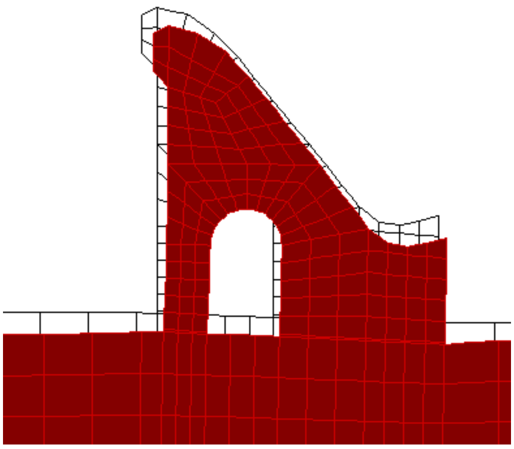
Stresses σ_1



Stresses σ_3



Stresses σ_2

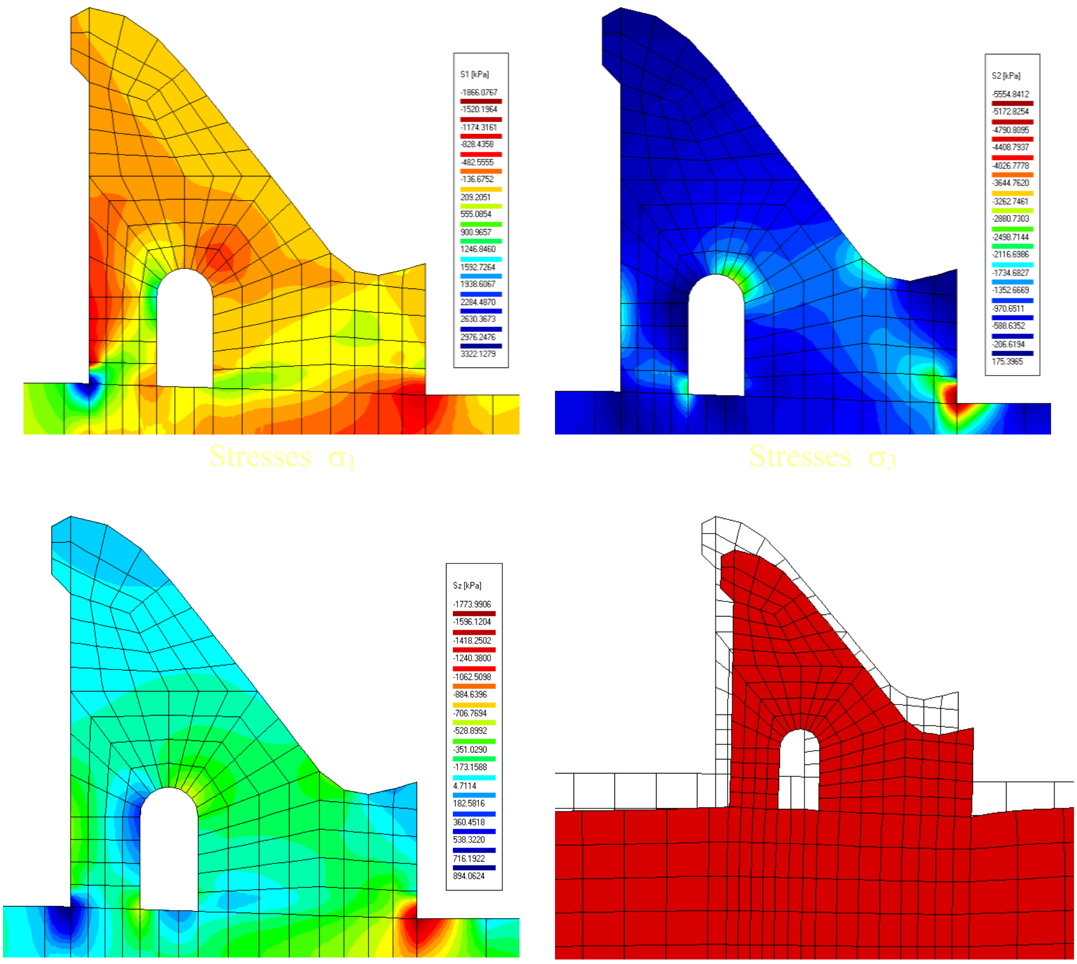


Deformed mesh

With the purpose of reducing the stress levels around the gallery and near the upstream bottom apron, the same calculations were performed for the suggested improved geometry and implantation of the arc-rectangle gallery at the RCC dam core, nevertheless satisfying the mentioned geometric and operational restrictions.

The following Figures 10 to 13 refer to such improved geometry, associated with a decrease of the gallery dimensions and a more downstream implantation in the RCC dam core, and illustrate substantial reductions in the values of the generalized quantities represented in the two distinct solutions.

Improved geometry and implantation



Stresses σ_1

Stresses σ_3

Stresses σ_2

Deformed mesh

4.4 Safety Analysis

With the knowledge of the distribution and values assumed by the principal stresses for the seismic load case, a verification of the safety of the dam has to be carried out at critical points based on a reasonable yielding criterion. Barros [10,11] used earlier a similar approach and methodology, when comparing less refined and established yielding criteria for concrete in dams under biaxial state of stress, nevertheless appropriate and conservative for preliminary design.

Herein the one proposed in the CEB-FIP Model Code 1990 [12] for biaxial stress state of concrete will be used, with the necessary characteristics of the RCC. Designate by $\alpha = \sigma_1 / \sigma_3$ the ratio of minimum to maximum principal stresses.

At failure *(f)* under biaxial state-of-stress the yielding criterion of concrete, represented in Figure 14, is expressed by

$$\textit{(Biaxial Compression)} \qquad \sigma_{3f} = -\frac{1+3.80\alpha}{(1+\alpha)^2} fck \tag{9}$$

$$\textit{(Traction} - \textit{Compression)} \qquad \sigma_{1f} = \left(1+0.80\frac{\sigma_3}{fck}\right) fctk \tag{10}$$

$$\textit{(Biaxial Traction)} \qquad \sigma_{1f} = fctk \tag{11}$$

where the characteristic values used herein for the compressive strength of RCC (after a cure of 90 days) and for the tensile strength of RCC are respectively $fck_{90} = 12\,MPa$ and $fctk = 1.1\,MPa$.

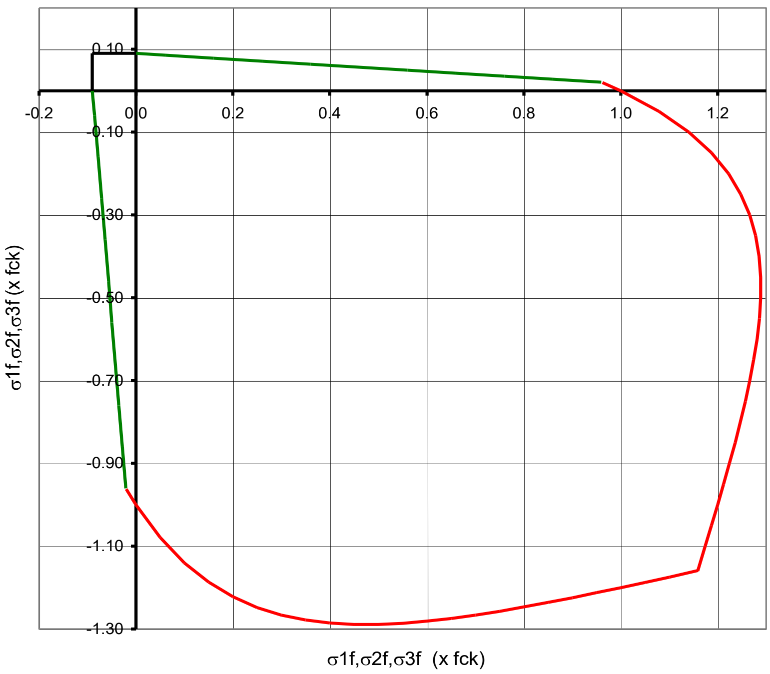


Figure 14: Yielding of concrete under biaxial state-of-stress

Figure 15 illustrates quite efficiently the comparison between the results of the yielding zones associated with the seismic load case, obtained here for the preliminary initial geometry tested (larger gallery) and for the improved geometry and implantation (smaller gallery towards downstream). It is evident from the relative comparison that the yielding stripe or traction band developing at 45º (for the initial geometry) between the upstream bottom toe and the gallery upstream side, is reduced to an incipient formation of a yielding zone due to stress concentration and almost disappearing thereafter (for the improved geometry).

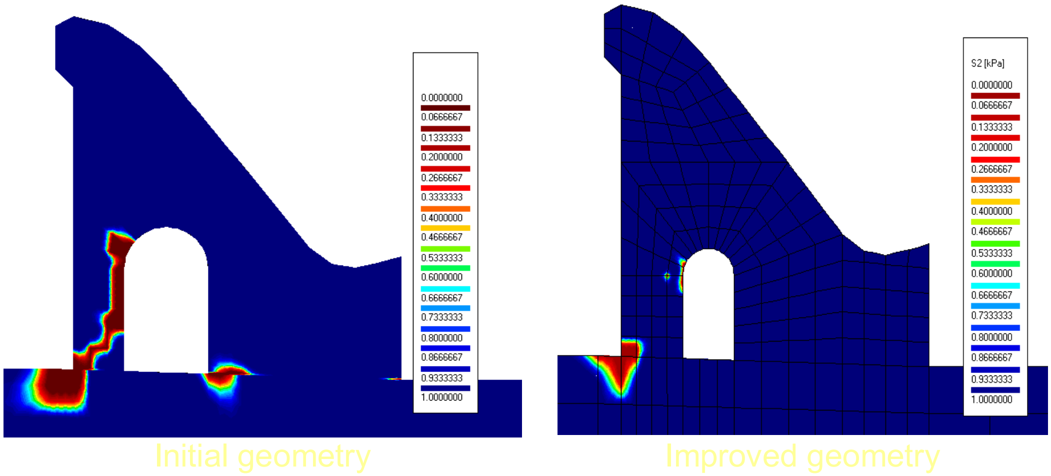


Figure 15: Yielding zones for the two different geometries

It should be emphasized that the potential yielding zones for the RCC dam are located at places where conventional structural concrete is adequately used, with strength characteristics far superior to the ones for RCC being used in the yield criterion. Therefore, the previous analyses are conservative estimates. Moreover, other adjustments on the gallery size and location can be efficiently accomplished satisfying other required restrictions.

In view of the fact that structural safety of the RCC is as important as the geotechnical safety of the foundation, an assessment was made of the stress patterns and magnitudes at various foundation levels. For the two foundation levels represented in Figures 16 and 17, the stress assessment along the surface (1-1') corresponds to the interface with the dam, while the other stress assessment occurs in a deeper foundation layer (2-2') around 15 meters below the reservoir bottom.

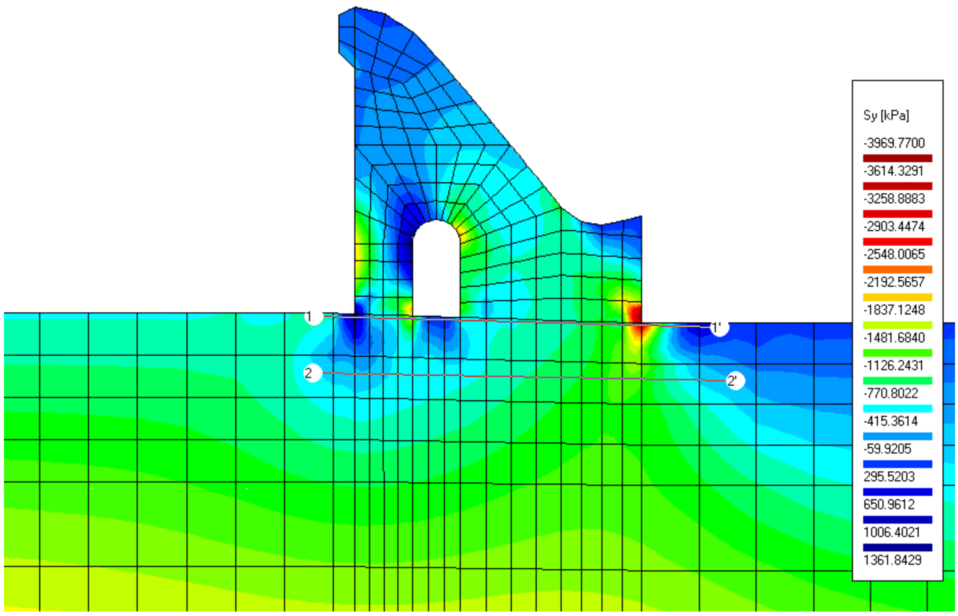


Figure 16: Distribution of vertical stresses in the dam-foundation finite block

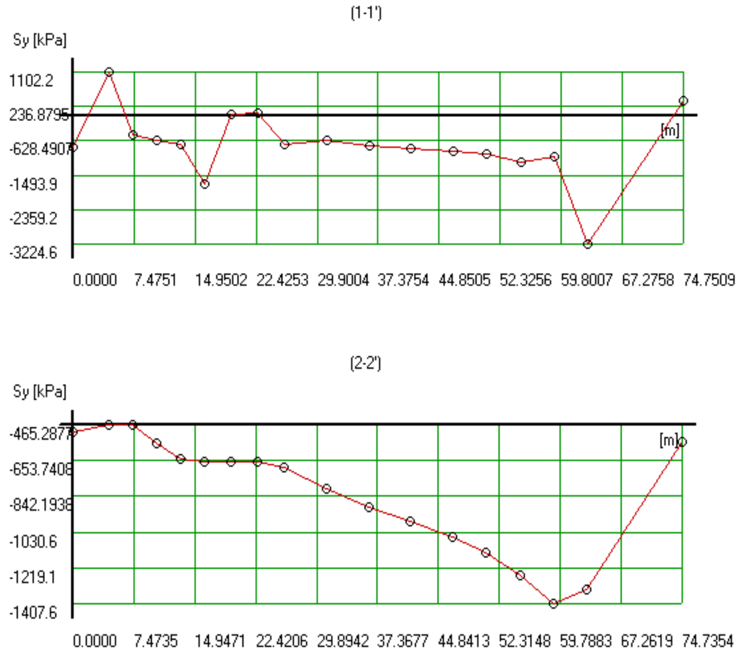


Figure 17: Vertical stresses (σ_y) on foundation, along two surfaces

In Figure 17, the trends and magnitudes of the vertical stresses along line 1-1' reveal the expectable influence of boundaries and discontinuities. However the trends and magnitudes of the vertical stresses along line 2-2' are much more realistic in terms of being able to describe the true vertical stress evolution across the foundation at this depth. The peek value of 1.4 MPa is quite safe in terms of the expected compressive strength values of the bed-layer at the site (around 10 MPa).

4 Conclusion

A preliminary design for a gravity dam to be built with roller compacted concrete, located in central Portugal, was thoroughly detailed. The international tendering and bidding for conception and construction was justified. Design aspects were emphasized related to the adoption of a gallery satisfying geometric and construction constraints, as well as aspects associated with safety of the structure and adjoining foundation. A preliminary analysis for the initial geometry proved structurally safe, under strength stiffness and stability criteria. A plane stress finite element analysis calculated accurate values of generalized stresses and displacements of the dam and foundation. A safety analysis of the biaxial state-of-stress was performed according to the CEB-FIP Model Code 1990, indicating the need for geometric refining insuring safety under a specific seismic load combination. The improved section for the dam with a smaller gallery positioned more downstream at the dam core, induced safe detailed patterns of stresses and displacements for the dam and foundation. The comparison of the yielding zones of the two studied geometries is an important tool to ascertain the safety of distinct dam solutions.

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