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THE FOZ TUA ARCH DAM DESIGN

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Abstract. *Portugal is presently facing a new hydroelectricity era, with the implementation of a large development program including not only the power upgrade of existing large hydro schemes, but also the construction of several new ones, almost all of them considered in the "National Program of Dams with High Hydroelectric Potential", dated of December 2007.*

EDP-Energias de Portugal was awarded the concession of three hydroelectric schemes included in this Program and Foz Tua Hydroelectric Project was the first one to be launched. The Project is located in the North region of Portugal, in the Tua River, an important tributary of Douro River, close to its confluence into Douro River. The feasibility study of this scheme was developed by EDP in 2008, which included a complete environmental assessment of the project envisaging the Authorities approval.

Actually the project has been submitted for approval with the defined Environmental Conformity Report and EDP has already received the contractor's proposals to the Project construction, which will start in 2011.

The project includes a double curvature arch dam, 108 m high provided with a surface controlled spillway over its crest designed for a 5500 m^3/s flood, an hydraulic circuit in the right bank, about 700 m long, composed of two independent tunnels and a downstream powerhouse equipped with two reversible units with a rated output of 131 MW each one.

After a general description of the project, the paper details the structural analysis performed to the dam design and to foresee the dam-foundation behavior according to the Portuguese Regulation requirements.

Furthermore, structural analysis carried out by the National Laboratory of Civil Engineering (LNEC) to assess the dam safety with respect to failure scenarios are also presented, namely involving concrete damage, foundation failure and the maximum design earthquake effects.

1 BRIEF DESCRIPTION OF THE PROJECT

The general lay-out of the project has been presented in the above abstract. The designed arch dam is a 108 m high structure, with a total crest length of 275 m at elevation 172 and a total concrete volume of 317 000 m^3. In the dam site, the valley is quite narrow and the rock foundation is sound granite. Downstream, close to the confluence with the Douro River, the valley becomes wider and schist mass rocks are present.

The catchment area at the dam site is of 3809 km^2, the mean annual precipitation is of 940 mm and the mean annual runoff is 1421 hm^3. For the full storage level located at elevation 170 m a.s.l. the reservoir capacity is of 106 million cubic meters. A controlled surface spillway, designed for a 5500 m^3/s flood, is located in the central part of the dam crest, provided of four 15.7 m wide spans controlled by radial gates and with a downstream plunge pool (Figures 1 and 2). The dam is also provided of a bottom outlet with a discharge capacity of 200m^3/s. A diversion tunnel located in the left bank and two concrete gravity type cofferdams will allow the dam construction in the river bed zone. Structural and hydraulic calculations have been made in accordance with Portuguese Rules on Dam Safety.

The geotechnical characteristics of the site are also favorable to the designed underground hydraulic circuit in the right bank of the valley entirely separated from the dam structures. The intake is located 100 m upstream the dam abutment with the crest located at elevation 140.50 m. The two independent pressure tunnels have concrete lined lengths of 551 and 635 m, and an internal diameter of 7.5 m. Close to the powerhouse the tunnels are steel lined, with internal diameter of 5.5 m and lengths of 67 and 71 m.

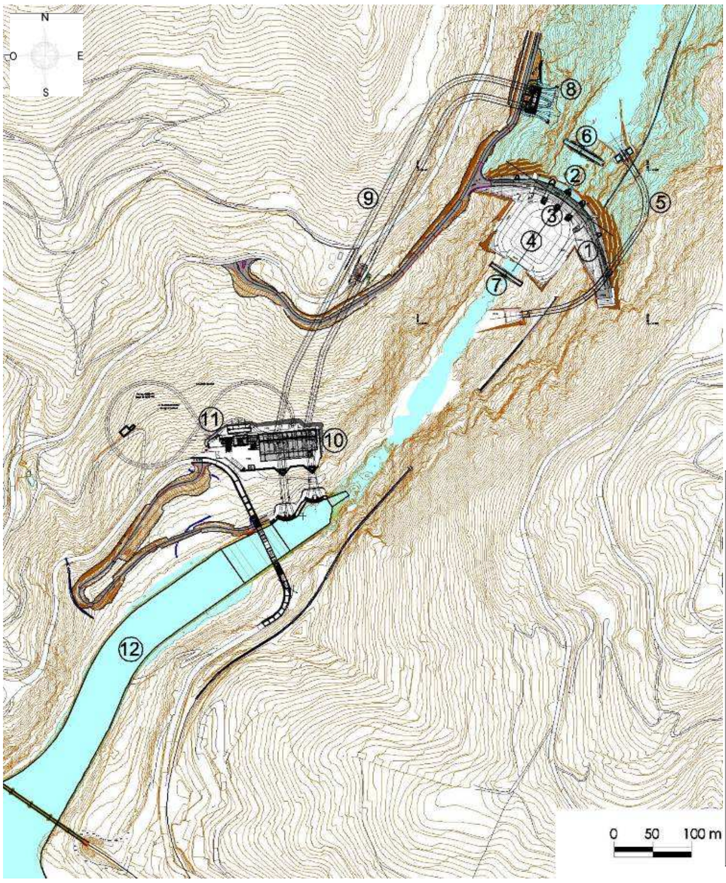

Figure 1: Lay-out of the Foz Tua Hydroelectric Scheme.

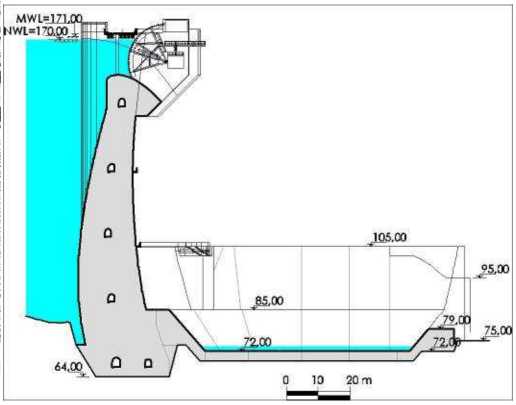

Figure 2: Cross section of the dam in the spillway zone.

The underground powerhouse is composed of three shafts connected by a cavern with several technical floors. The two shafts, where the Francis type turbine-pump units are located, are 67 m deep and their internal diameter is of 13 m. The access and bus-bar shaft is 82.5 m deep. For each unit the gross head is of 96 m and the rated output is of 131 MW. Power generation simulations carried out for different scenarios made possible to conclude that the power generation will be between 577 and 669 GWh/year and the net pumping generation between 282 and 285 GWh/year.

The two independent tail race tunnels will have lengths of 83 and 103 m, and the outlet structures will be located about 600 m upstream the confluence of the Tua River into the Douro River. The excavation of the downstream zone of the river bed, necessary to enable adequate pumping conditions, requires the execution of specific works to protect the foundation structures of two bridges over Tua River: a road bridge located near the outlets structures and a railway bridge located close to the confluence between Tua and Douro rivers.

2 MAIN FEATURES OF THE ARCH DAM DESIGN

As already mentioned, in the dam site the valley has a canyon shape and granite sound rock is present with outcrops visible all around (Figure 3). The main sets of rock joints are vertical and their orientations are approximately parallel (N20-30ºE) and normal to the river axis. At the river banks surface the joints are opened and origin blocks with considerable dimensions.

Figure 3: Upstream (left) and downstream (right) views of valley at the dam site.

Geological investigations were carried out in two phases, the first one during the Feasibility Study and the second one during development of the Design submitted for official approval. It must be pointed out that in both phases the geological investigations had severe restrains associated to the ecological value of the dam site and to the submersion of a significant part of a centenary railway by the future reservoir.

The main investigation works included geophysical survey and the execution of 8 trenches, 4 inspection galleries and 20 boreholes. The foundation geotechnical properties were estimated based on the results of dilatometer tests carried out in the boreholes, of large flat jacks tests performed inside the galleries, and of laboratory tests on specimens from the boreholes. Results of compressive strength, deformability modulus and sliding on rock joints were the most important ones to evaluate the geotechnical foundation parameters which were considered in the dam design.

According to the obtained results, the dam was designed to have its foundation mainly in rock mass with a quality index of W1-W2 – compressive rock strength of 130 MPa and a rock mass deformability modulus of 32 GPa. A F1-F2 index was associated to the discontinuities classification. The faults that cross the dam foundation are not very important and only usual localized treatments are foreseen.

At the valley bottom, both the excavation’s deeps and the arches lengths had to take into account the downstream plugging pool of the surface spillway, and so they assume values bigger than those strictly required due to the geotechnical conditions.

The adopted shapes of the arch dam were based on parabolic arches and were defined by the following functions: the crown cantilever axis and thickness, the curvature radius of the parabolic arches axis at the crown cantilever and the increasing evolution of the arches thickness to the abutments. Polynomial definitions were used for these four functions, where the depth measured from the crest level is the independent variable. Since some decades this type of definition has been used by the Department of Dams of EDP (Energias de Portugal, S.A.) in the design of several arch dams. The main characteristics of the last projects are summarized in the following table, where the figures of the Foz Tua dam are included.

	Height (m)	Length (m)		Thickness (m)			Concrete volume (m^3)
		Crest	Bottom	Crest	Basis of crown cantilever	Maximum at arches abutment	
Alto Lindoso	110	297	60	4.0	21.0	30.1	295 000
Alqueva	96	348	145	7.0	30.0	33.0	527 000
Foz Coa	136	434	80	6.0	37.7	43.4	972 700
Fridão (1993)	96	300	32	4.5	26.7	27.6	227 800
Baixo Sabor	123	505	60	6.0	27.0	39.3	670 000
Foz Tua	108	275	68	5.0	22.0	32.0	317 000

In Figure 4 the crown cantilever shape of the above-mentioned dams are shown.

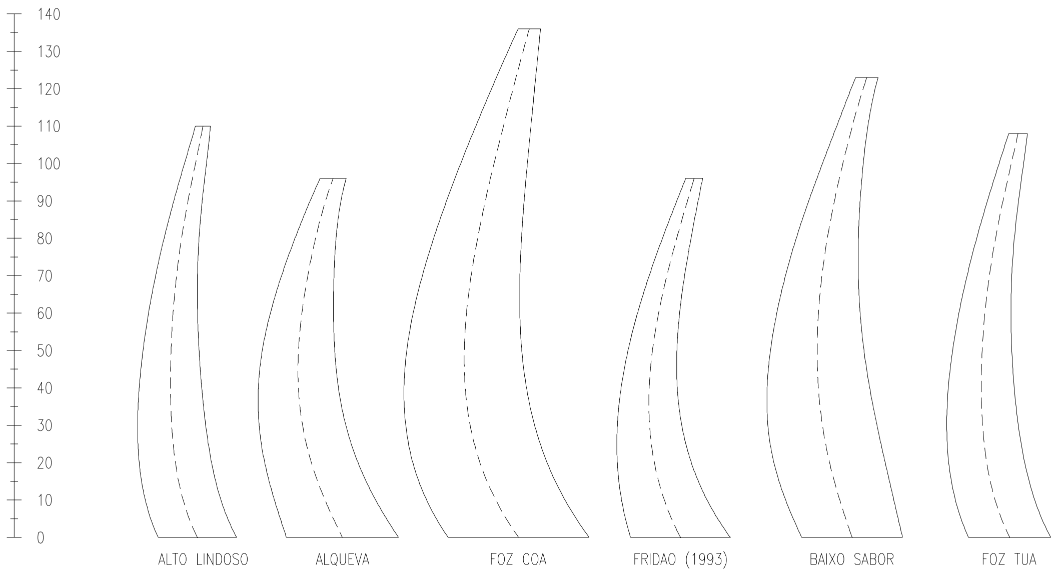

Figure 4: Shape of arch dams crown cantilevers.

Preliminary design is usually based on the acquired experience and on the comparison with similar projects. Then, the stability of the crown cantilever section for dead weight action and the optimum crown curvature for the parabolic arches, in accordance with the valley morphology and the geotechnical conditions, are taken into account. In this stage, analysis using the trial-load method is yet a very valuable way for the first shape adjustment.

In a second stage, shape definition is refined using finite element models and taking into account the main static and dynamic actions. Subsequently, a shape optimization process is usually carried out, in order to reduce the concrete dam volume enforcing an appropriate structural behavior.

The limit values of 5.5 and 1.0 MPa for the compression and tensile stresses, respectively, were considered in the following loading scenarios: construction phase and reservoir filled up to the normal water level, with and without winter thermal action. The corresponding values for the scenario of empty reservoir in the summer period were 7.0 and 1.0 MPa.

The limit values of 0.85 and 0.95 were adopted for the ratio between the tangential and normal forces at the base of each dam block for the scenario of reservoir filled up to the normal water level, respectively with and without winter thermal action, and having in account the uplift pressures.

After all, a final checking of the structural behaviour of the dam for all the mandatory scenarios according to Portuguese regulations on dams[2] was performed.

3 STRUCTURAL ANALYSIS FOR EXPLOITATION SCENARIOS

The detailed structural behavior of the dam-foundation structure was foreseen by finite element analyses performed with the Ansys software. The adopted finite element mesh model of the dam body and the adjacent foundation zone was composed of 1572 three-dimensional hexahedral finite elements, each with 8 nodes, and 2790 nodes (Figure 5). Four elements were set along the thickness of the dam.

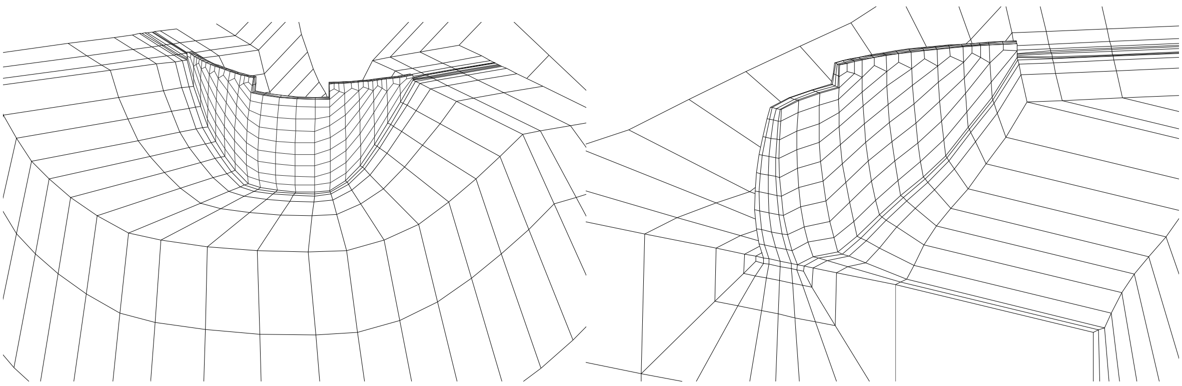

Figure 5: Finite element mesh.

The relevant load combinations, considered according to the Portuguese Regulation requirements, comprise the dead weight of the independent dam blocks along construction ($\gamma = 24$ kN/m^3), the hydrostatic pressure ($\gamma = 10$ kN/m^3) at the upstream face for the characteristic water table levels of the exploration of the reservoir, the extreme fields of the seasonal variations of temperature with respect to the reference field occurring at the injection of the contraction joints, and the basis design (BDE) and maximum design (MDE) earthquakes.

The concrete and foundation mass were considered as isotropic, linear elastic materials with the values E = 20 GPa, $\nu = 0.2$ and $\alpha = 10^{-5}$ ºC^{-1} for the mechanical proprieties.

The structural behaviour is nearly symmetrical with respect to the crown cantilever, and agrees in general terms with the above limits prescribed in the optimization phase. It is pointed out that, as a result of this phase, for several load combinations the extreme values of the prescribed stresses occur in vast zones, showing a good exploitation of the structural material.

These aspects are illustrated in Figure 6 which shows the principal stresses at the upstream and downstream faces, for the scenario of the reservoir filled up to the normal water level without thermal action (0.9 MPa tensile and 5.0 MPa compressive stresses along the insertion of the upstream and downstream faces in the foundation, respectively). The raise of the compression arch stresses (with values 3.3 MPa, both upstream and downstream) just below the spillway opening in the central zone of highest arches is noticeable. Figure 7, which represents the principal stresses along the crown cantilever, points out the decrease of the compression stresses along the superficial layer of both faces, promoted by the winter thermal action.

The evaluated behaviour for the scenario comprising the BDE seismic action, which was defined by a response spectra based in the Portuguese Regulation and a 0.029g peak ground acceleration (return period of 145 years), is also within the linear elastic range of the concrete.

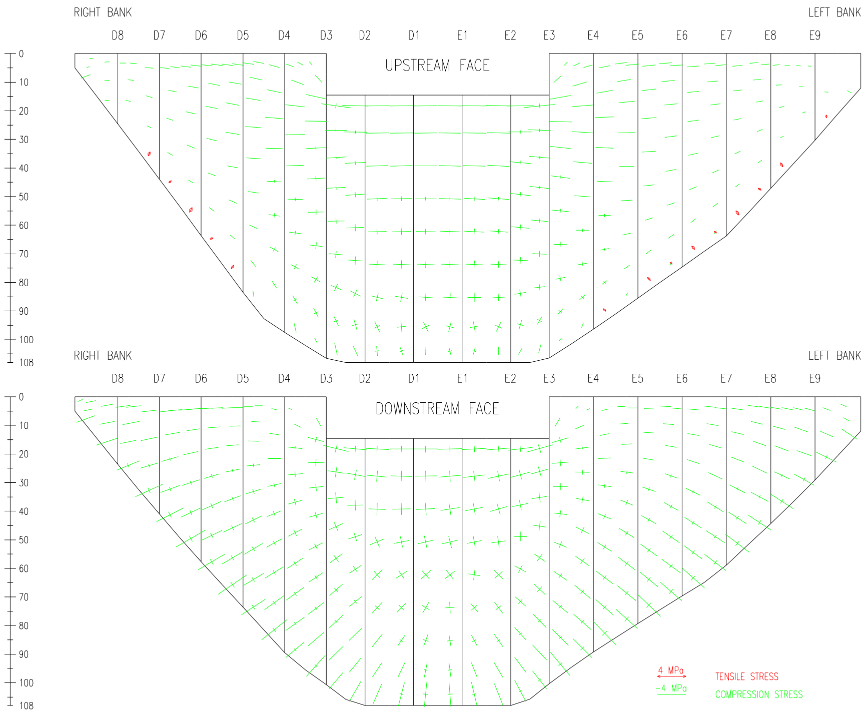

Figure 6: Principal stresses for dead weight and hydrostatic pressure.

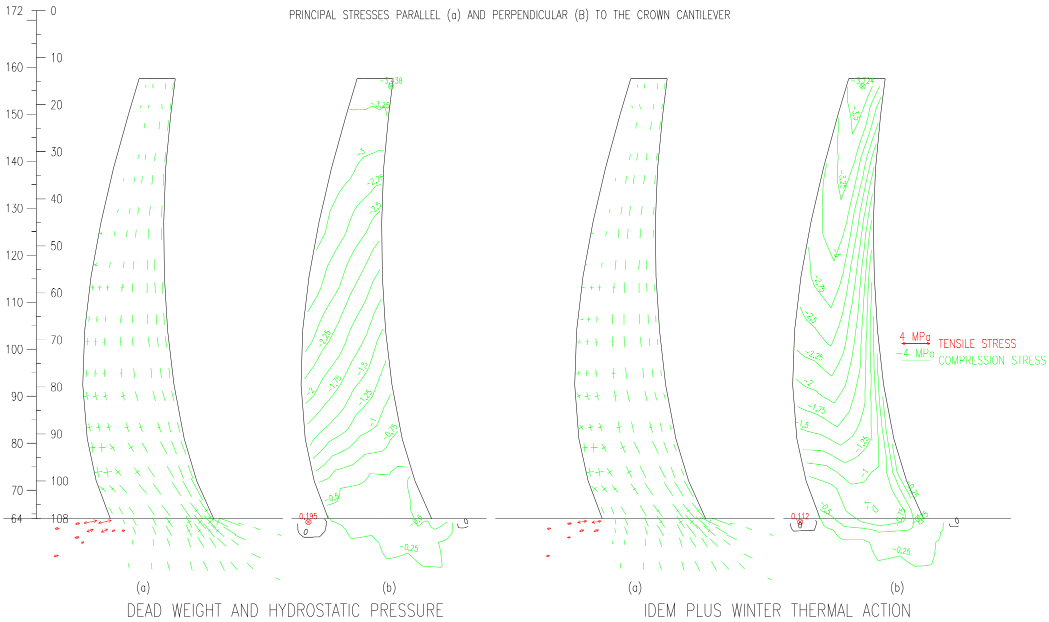

Figure 7: Principal stresses along the crown cantilever.

4 STRUCTURAL ANALYSIS FOR FAILURE SCENARIOS

The National Laboratory of Civil Engineering (LNEC) performed several studies for the safety assessment of the dam-foundation structure with respect to the following failure scenarios: concrete arch failure[3], maximum design earthquake[4] and foundation failure[6].

4.1 Concrete arch safety assessment

For many years, the assessment of the concrete arch dam designs was performed by means of physical models. Models were loaded with jacks up to failure, typically increasing simultaneously both dead weight and hydrostatic loads, leading to a global safety factor. Numerical models are now capable of simulating the material failure process. A safety measure comparable to the traditional one given by experimental test may be obtained by increasing the numerical model gravity and water loads. A similar safety factor may also be obtained by progressively reducing the concrete strength until structural collapse ensues, and therefore this failure scenario is commonly referred to as the concrete deterioration scenario. In this analysis, the concrete behavior was represented by an isotropic continuous damage model, in which the weakening is governed by two independent damage variables, one for tensile and the other for compressive failure[3].

Numerical analyses were performed for six sets of values of the parameters of the concrete behavior law, namely Young's modulus, uniaxial compressive and tensile strength, fracture energy and ultimate failure strain under compression. Compressive failure of the arch is the key element in this scenario, so the values of uniaxial compressive strength considered, 20 and 30 MPa, bracket the expected strength range, while changing the ultimate strain provides a means to assess the influence of the softening strain slope. The finite element method was used with a mesh of three-dimensional second order isoparametric finite elements with 20 nodes (Figure 8). The foundation was considered homogeneous, without discontinuities, with an isotropic linear elastic behavior.

The evaluated collapse of the structure, achieved by magnification of the loads, was characterized by a usual mechanism for this type of structures (Figure 9), consisting in the crushing of the upper central zone. This failure mode takes place following the appearance of tensile cracking near the foundation in the upstream face and the compression failure at the downstream face near the abutments, where high compression stresses occur perpendicularly to the insertion surface.

The global safety factors obtained, ranging from 6.3 to 10.8, depending on the values of the parameters adopted for the concrete, are values expected for this kind of structure.

5.2 Dynamic analysis for the maximum design earthquake

The seismological study undertaken for this specific site led to a Maximum Design Earthquake with a peak acceleration of 0.27g, and a BDE of 0.029g. Two sets of time series were generated for each scenario, to be used in time domain dynamic analyses. In each run, the seismic excitation was applied in 2 directions, upstream-downstream and vertical, with the vertical input being scaled buy a factor of 2/3.

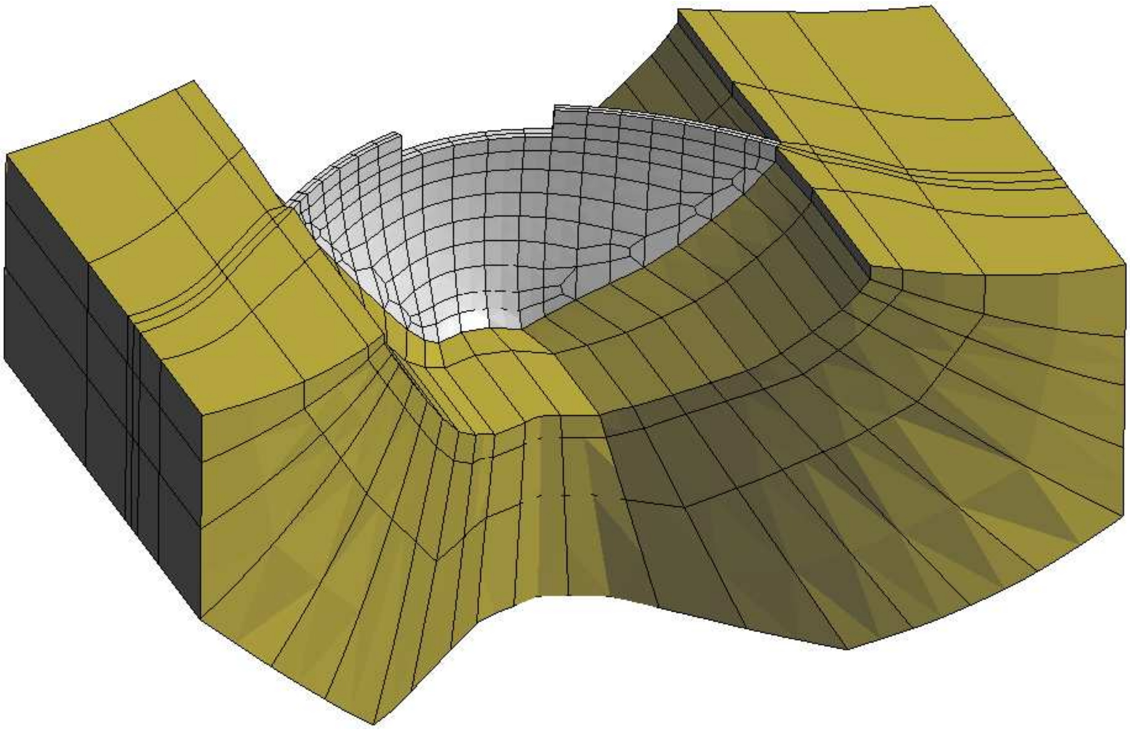

Figure 8: Concrete arch failure model. Finite element mesh.

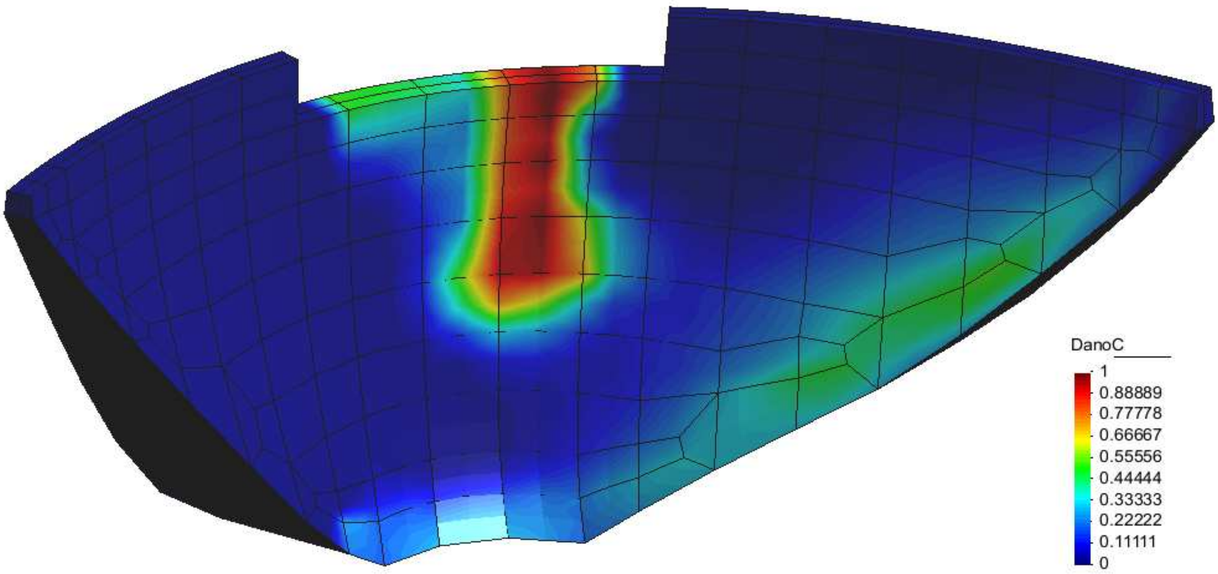

Figure 9: Concrete arch failure model. Contours of compressive damage variable near the structural failure condition.

The numerical model[4] considered the dam body as a group of elastic blocks, separated by contraction joints with purely frictional behavior and no tensile strength. The concrete-rock interface was assumed to be governed by a Mohr-Coulomb joint model with limited tensile and cohesive strength. The foundation rock was idealized as an elastic and isotropic continuum. The analysis was conducted with the code 3DEC, however, as the nonlinear behavior was restricted to these discontinuities, 20-node brick finite elements were used for the dam and rock mass meshes. The application of the seismic input followed the general scheme developed for blocky rock foundations, assuming upwardly propagating stress waves with lateral numerical free-field conditions and a bottom non-reflective boundary condition[5], as shown in Figure 10. The hydrodynamic interaction was represented by Westergaard added masses. Rayleigh damping was assumed for the dam, with 5% at the fundamental frequency of 2.7 Hz. No damping was considered in the rock mass.

The set of analyses performed for the MDE led to maximum compressive stresses in the concrete arch elements of 9 MPa, and maximum tensile stresses of 3 MPa. The contraction joints remained mostly in compression, with only a few episodes of opening near the crest (Figure 11), with a maximum separation in the order of 2 mm. The foundation joint displayed nonlinear behavior in a few locations at the upstream edge. It is therefore expected that, under this level of seismic action, the dam blocks will remain mostly in the elastic range, with limited tensile cracking in the vicinity of the foundation joint.

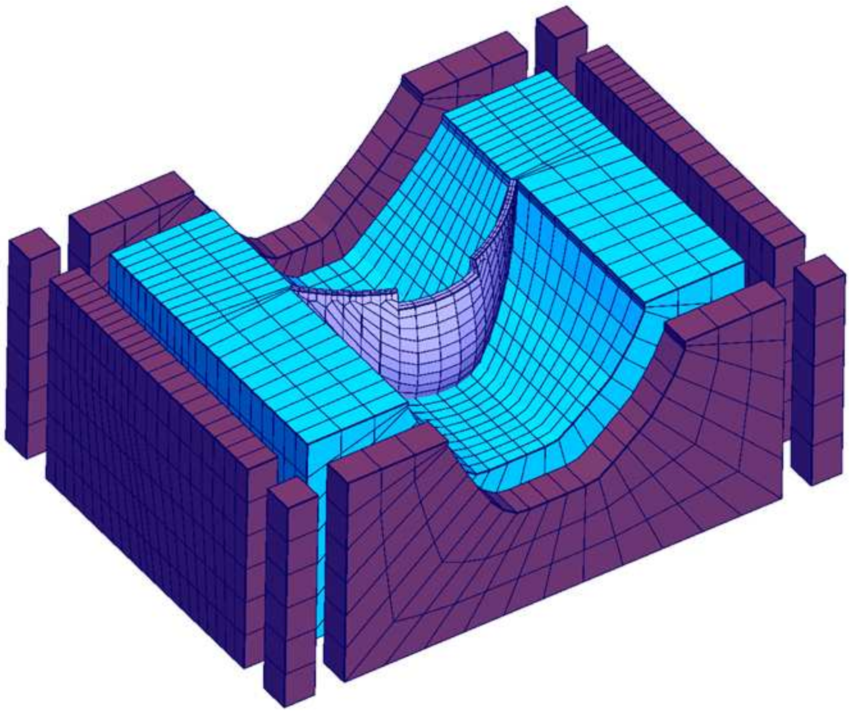

Figure 10: 3DEC model for seismic analysis. Dam with contraction joints and foundation joint, rock mass and lateral free-field meshes.

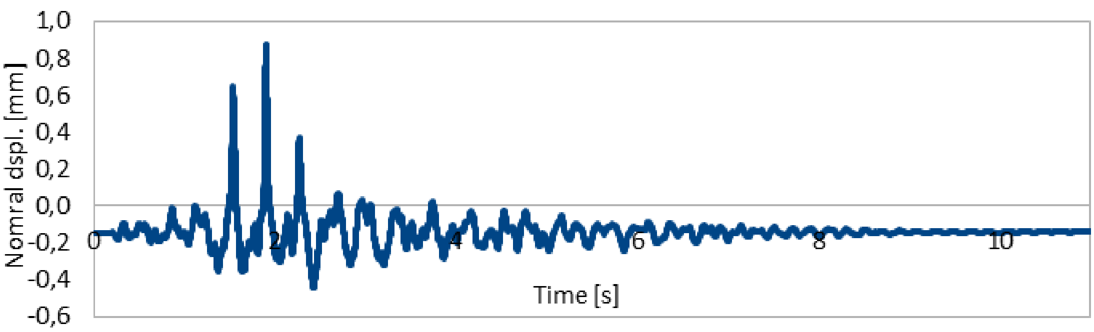

Figure 11: Normal displacement of a contraction joint at the crest (positive is opening).

4.3 Foundation failure

The assessment of the static stability of the dam and foundation along surfaces of weakness was performed through several analyses with the discrete element software 3DEC, which models the dam and the rock foundation as system of deformable blocks[5]. Besides the rock mass faults and joints, the contraction joints in the dam body and the concrete-rock interface are also discontinuities which may be involved in the failure mechanisms. The mesh used for the dam itself is actually the same one used in the dynamic analysis, shown in Figure 10. In the present model, however, the discontinuities in the rock mass are also included. The foundation block is therefore cut by a series of planes corresponding to faults and joints, leading to a system of polyhedral blocks which are then automatically discretized into a tetrahedral finite element mesh. In all the discontinuity surfaces, a joint behavior governed by a Mohr-Coulomb criterion is assumed. Water pressures, according to the usual design assumptions, are prescribed in all the discontinuities.

Three different models were created to study the foundation failure scenarios of Foz Tua dam[6]: a first model assuming an elastic rock mass, to study failure only through the foundation surface; and two more elaborate models, for the left and right banks, considering also failure through the rock discontinuities. Figure 12, where the dam and the upstream rock are hidden for clarity, shows a detail of the left bank model, with a block structure created by the main faults, placed at their known locations, and a few joint planes representative of the main joint sets. Static stability was checked by gradually reducing the strength characteristics of the joints until failure occurred or displacements reached unacceptable magnitudes.

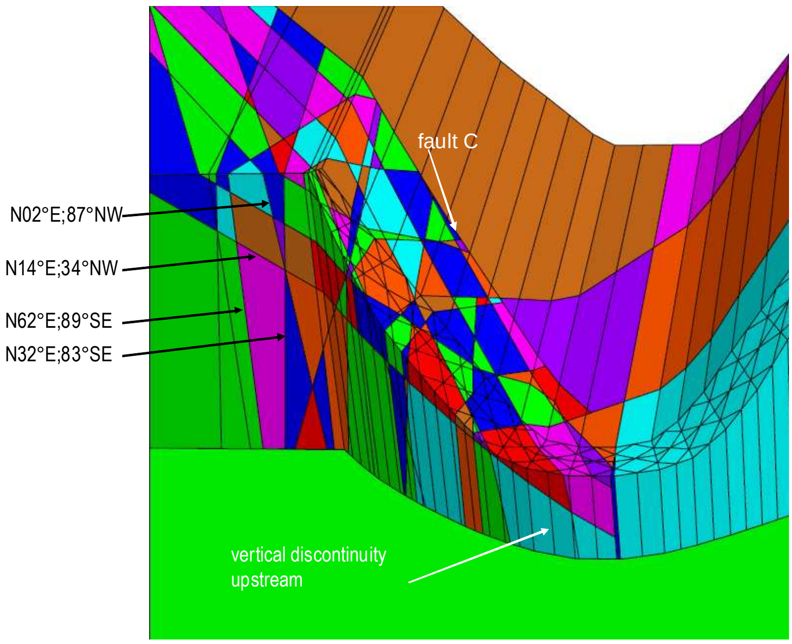

Figure 12: Detail of 3DEC model for the analysis of failure of the left abutment (dam and upstream rock hidden).

5 CONCLUSIONS

In this paper, the main aspects of the design process of the Foz Tua Arch Dam are presented. It is described the studies carried out envisaging the shape definition of this arch dam, which is 108 m height and 275 m long at the crest. The structural analysis performed by EDP for the exploitation scenarios and the analysis performed by LNEC for the failure scenarios show a good structural behavior of the dam in accordance with Portuguese Regulations requirements. Furthermore, the total volume of the dam shows that an economical design has been achieved.

ACKNOWLEDGMENTS

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The failure scenario models were performed by LNEC research officers Noemi Schclar Leitão, Sérgio Oliveira and Nuno Azevedo.

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