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The reshaping during construction of the right abutment foundation of the New Alto Ceira Dam in Portugal

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Abstract

The New Alto Ceira Dam is a 41 m high and 133 m long at the crest concrete arch dam, located in the middle zone of Portugal. The objective of the dam's construction is to replace the existing dam located about 200 m upstream, which is highly deteriorated due to swelling phenomena. The New dam's foundation lies over sound schists, in which some sets of discontinuities can be observed.
During the dam construction, two expressive rock discontinuities (one on each bank), that could cause problems to the dam-foundation safety, were identified, leading to additional studies of foundation failures scenarios. The discontinuity in the right bank was parallel to the right abutment surface and about 3 m deep. Under these conditions, the foundation of the abutment was redesigned.
The 3DEC (3-Dimensional Distinct Element Code) was used to simulate the potential failure mechanisms defined by the natural rock discontinuities and the concrete-rock interface in order to evaluate the safety and effectiveness of the proposed foundation design. The numerical model represents the geological structure of the rock mass, the concrete vault with its contraction joints and foundation surface. The actions of the dead weight, reservoir water pressure, uplift pressures and water pressures in rock were taken into account. The safety analysis was performed by progressive reduction of strength along the discontinuities until the failure condition was reached, and the safety of the right abutment for the new deeper surface foundation could be proved.
keyword: arch dam, rock foundation discontinuities, numerical models, discrete elements

1 Description and objectives of the New Alto Ceira Dam

The existing Alto Ceira dam is located in the central region of Portugal, and was built in 1949 to derivate water from the Ceira River and its tributaries into the Santa Luzia dam's reservoir. The deviated water is turbined not only in the Santa Luzia powerhouse but also in Cabril, Bouçã and Castelo de Bode hydro-plants located in the Zêzere River.

The Alto Ceira dam is a thin arch defined by circular arches of constant thickness, with a maximum height of 37 m. The central cantilever thickness varies between 4.5 m in the base and 1.5 m at the crest, and the dam crest has a length of 120 m. The dam is provided with a surface spillway with a maximum capacity of 200 m^3/s in the right bank.The diversion tunnel to the Santa Luzia reservoir is located on the left bank and has a maximum capacity of 9.0 m^3/s.

The first evidences of structural anomalies of Alto Ceira dam were observed during the first filling of the reservoir, and have increased since then. The anomalous dam behaviour is characterized by progressive horizontal upstream displacements, progressive upward vertical displacements and intensive cracking. Many studies and tests have been made to determine the causes of the structural problems. Alkali aggregate reactions (AAR) in the concrete have been identified as the main cause for the intensive cracking of the thin arch dam structure.

In 1994, after assuming the dam ownership and due to the severe structural anomalies of the dam, Energias de Portugal (EDP) decided to study, not only the dam's rehabilitation but also the construction of a new dam, close to the existing one.

Later on, in 2006, several additional solutions were studied envisaging at first, the dam rehabilitation and, on a second stage, the conception of alternative scheme configurations. The best reliable achieved solution, considering economical and environmental restrictions, was similar to the one reached back in 1994, and consisted in the replacement of the existing dam building a new one about 200 m downstream (Figure 1).

The new Alto Ceira dam is a concrete arch dam with gravity abutments. It has a 100 m crest length at elevation 668.50 a.s.l. and a 41 m maximum theoretical height above the foundation level. The arch zone of the dam comprises six blocks separated by vertical construction joints, 16.0 m apart between the three central blocks and 17.0 m on the other joints (Figure 2). Its shape definition is based on parabolic arches, with increase in thickness towards the abutments; the central cantilever theoretical thickness is 2.0 m at the crest and 5.5 m at the base (Figure 3); the dam's maximum thickness is about 6,5 m in each bank. The total concrete volume of the dam is about 17 000 m^3.

The right bank gravity abutment is 20.0 m and has a maximum height of 10,1 m. The correspondent dimensions of the left abutment are 13.0 m and 11.0 m, respectively.

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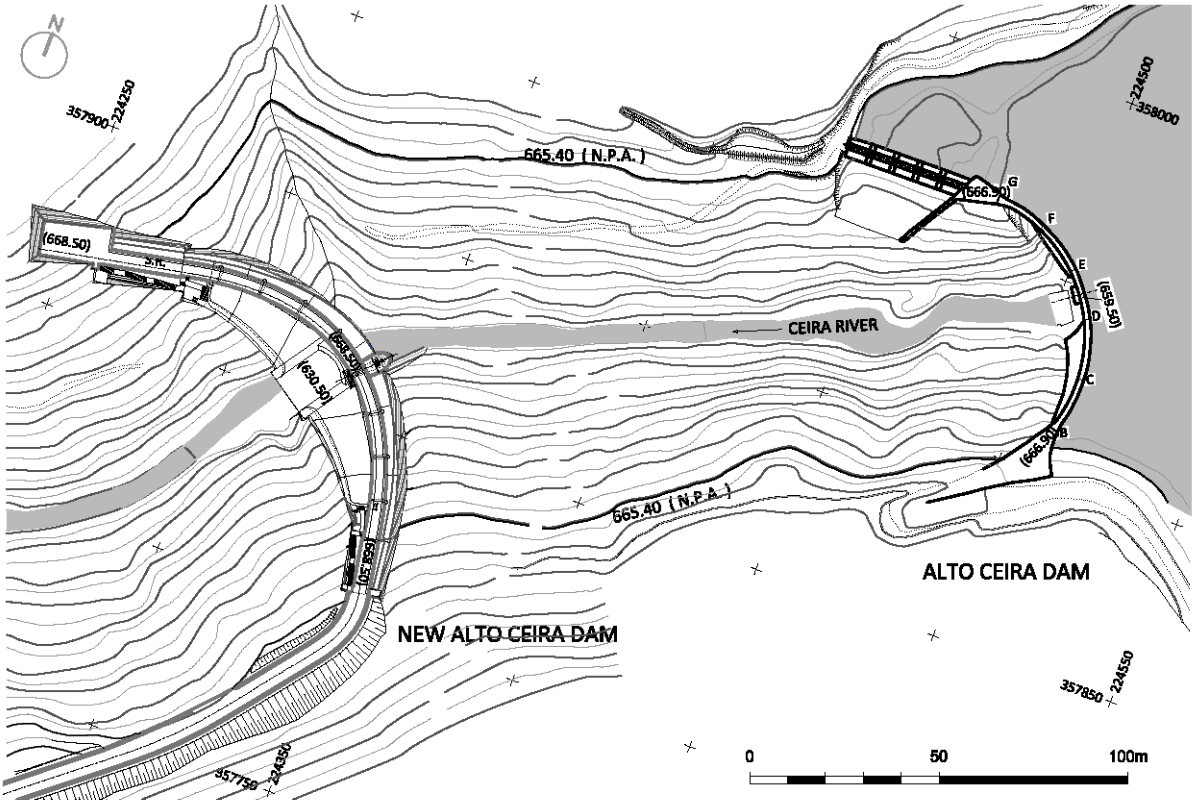

Figure 1 – The new Alto Ceira dam (left) and the upstream existing one (right)

The dam is provided with a surface uncontrolled spillway along almost the whole crest of the dam and with a downstream stilling basin. Its maximum discharge capacity is of about 200m^3/s. The dam is also provided with a bottom outlet, located in the dam´s central zone, with a discharge capacity of 15m^3/s (Figures 2 and 3). A device for environmental flows release will be also installed in the dam.

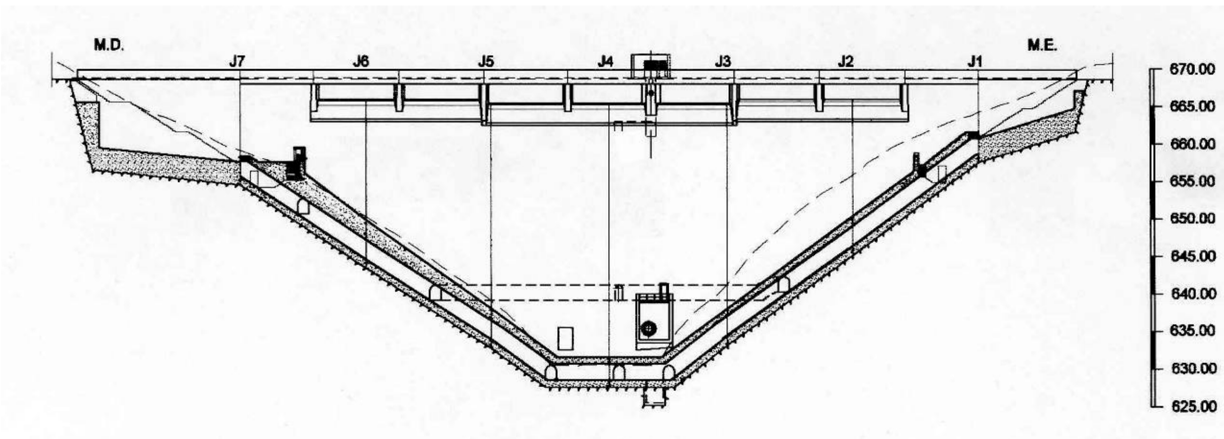

Figure 2 – Developed longitudinal section of the dam (downstream view)

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The dam monitoring plan is detailed in another paper of this conference. The prevention of AAR was a main issue of the dam design. In addition to a careful selection of the quarries that will provide the concrete aggregates, supported by the execution of reactivity tests, high percentage fly ash concrete mixes were adopted.

The construction of the new dam started in 2009 and will be completed in the end of 2012. Figure 4 shows a downstream view of the dam during construction.

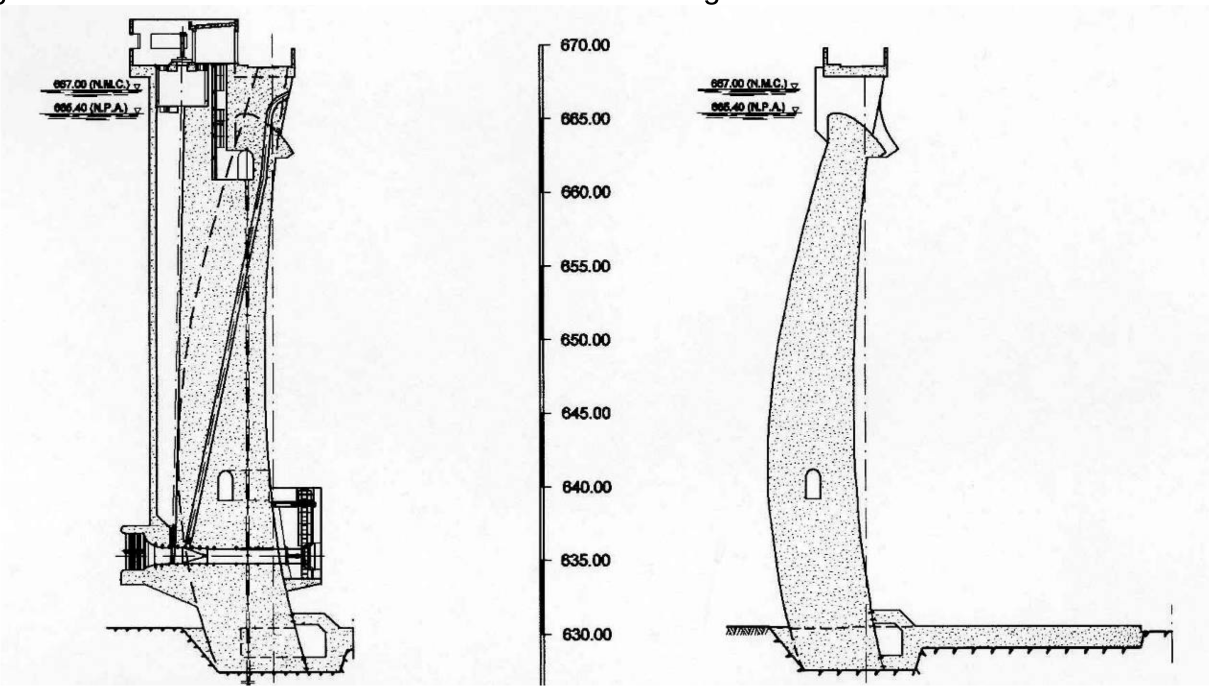

Figure 3 – Cross sections: bottom outlet (left) and central cantilever (right)

Figure 4 – Downstream view of the New Alto Ceira dam during construction

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2 Geological and geotechnical foundation conditions

2.1 Geomorphology

At the dam site, river Ceira flows in a N33ºE direction slightly asymmetric valley. Above elevation (646), the valley's average inclination is 32º on the right bank (RB) and 34º on the left bank (LB). Between the (646) and (634) elevations, the valley is narrower, with 39º on the RB and 43º on the LB. The river bottom is 4m wide at elevation (631) and the river channel walls inclination is 47º (RB) and 67º (LB) (Neiva, 1993).

The last episodes of valley incision are associated with the sea (base) level drawdown that occurred during the last glaciation period (Wurm), 110 kyear to 10 kyear before present, in the Pleistocene epoch.

2.2 Geological and geothecnical site characterization during design phase

A flysch type turbiditic sequence of dark grey phyllites (predominant) and fine grained metagreywackes of upper Pre-Cambrian to Cambrian age outcrop at the dam site. Black graphitic phyllite layers and, at the left bank, a small doleritic intrusive dyke also crop out. These rock formations show tight isoclinal folding (Neiva, 1993) with bedding N50º-70ºW, 73ºNE-90º (direction and dip intervals) and coplanar axial planes and schistosity, developed at the Hercynian's orogeny folding phases.

During dam site geological mapping, 4 sub vertical and 3 sub horizontal joint sets and 2 sets of faults (N16º-26ºW, 71ºENE-90º and N69ºW, 76ºSSW-90º) with gouge filling thickness <0,60m, more frequently <0,10m (Neiva, 1993), were observed and characterized.

In the early 90´s, EDP planned and contracted an extensive geological and geotechnical exploration program that started with mechanical exploration works consisting in the excavation of 5 trenches sub parallel to the valley (2 at the RB and 3 at the LB). These trenches allowed the execution of detailed geological mapping (Neiva, 1993), 10 exploration rotary boreholes (Teixeira Duarte, S.A., 1992) with a total length of 272,3m, and 79 permeability Lugeon tests as drilling progressed.

In sequence, dilatometer tests (LNEC, 1992) for rock mass "in situ" deformability characterization were executed inside these boreholes. The obtained dilatometer modulii values in S2 and S4 boreholes ranged from 5 GPa at the more superficial zones and 10-25 GPa at greater depths. For the S6 borehole (RB), the values obtained were considerably lower, ranging 2 to 5 GPa.

Seismic tomography tests between boreholes were also performed by LNEC (1992) allowing seismic velocity rock mass zoning, that consisted in 3 compression waves velocity (VL) horizons: A) a deeper horizon, below 10-15m at the RB (4150 < VL < 6330 m/s) and below 10-20m at the LB (5600 < VL < 8180 m/s); above that one, 2 shallower

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horizons were detected with very different velocities, B) VL < 1930 m/s at the RB and C) VL < 4330 m/s at the LB.

LNEC (1992) also performed a comprehensive set of laboratory tests that completed the geotechnical characterization of the foundation rock mass. These consisted of core samples joint tests (normal load and direct shear strength) and core samples rock resistance tests, such as uniaxial compressive strength (UCS), point load and ultrasonic wave tests.

An overview of some of the most relevant results of these tests is listed below:

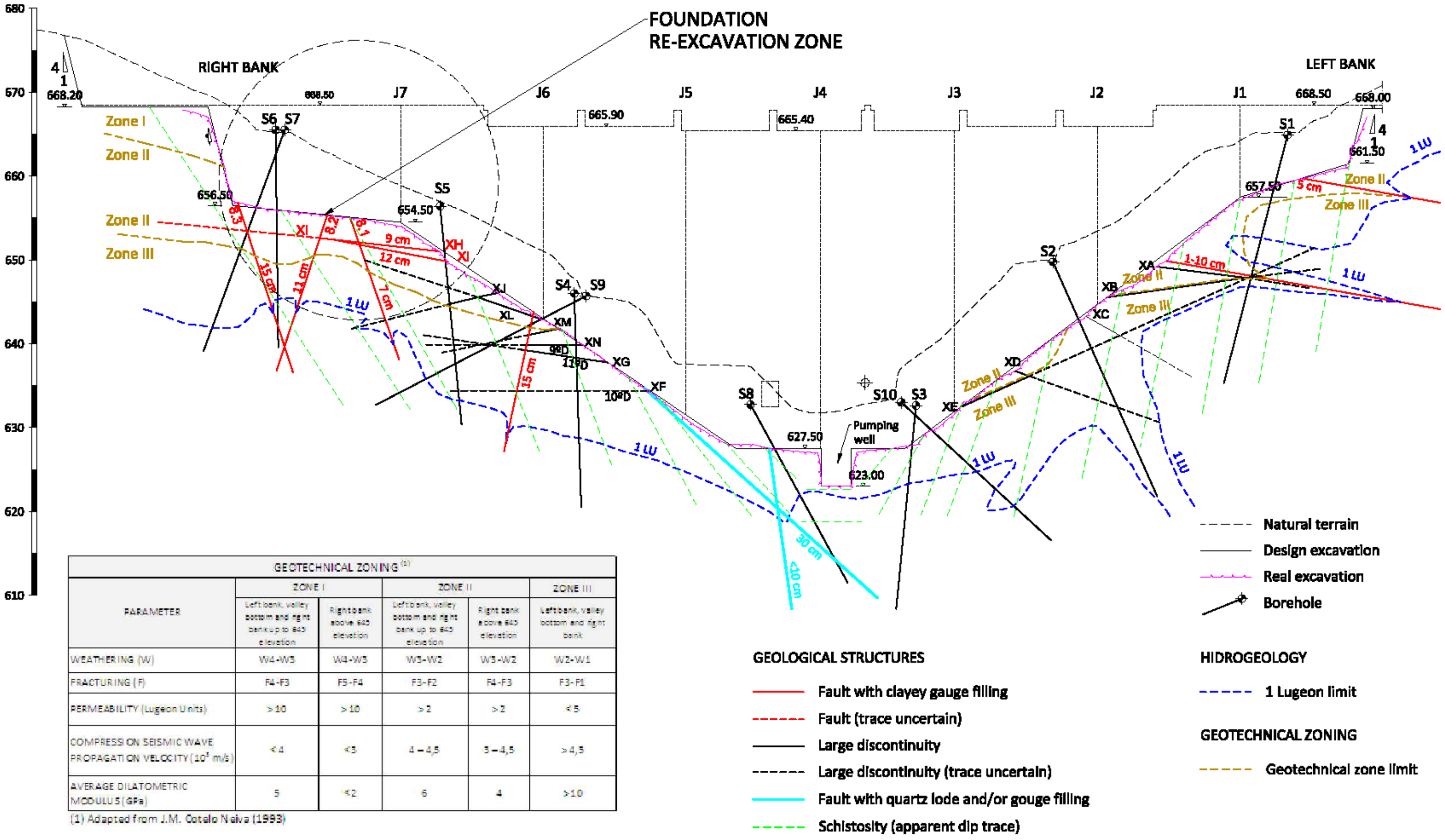
- Direct shear tests: (constant stiffness)
  - J1 (// to schistosity) $c = 60$ kPa, $\phi = 33{,}4^o$, $K_t = 2{,}57$ MPa/mm
  - J2 (sub vertical) $c = 62$ kPa, $\phi = 38{,}3^o$, $K_t = 3{,}12$ MPa/mm
  - J3 (sub horizontal) $c = 42$ kPa, $\phi = 38{,}6^o$, $K_t = 2{,}65$ MPa/mm
- UCS tests: $42{,}3 \leq \sigma_c \leq 116{,}5$ MPa (S2 and S4); $12{,}5 \leq \sigma_c \leq 59{,}7$ MPa (S6)
  $58{,}3 \leq E \leq 133{,}3$ GPa for S2 and S4; $57{,}5 \leq E \leq 86{,}2$ GPa (S6)
- Point load tests:
  - // to schistosity $0{,}17 \leq I_{S(50)} \leq 2{,}26$ MPa
  - $\perp$ to schistosity $1{,}30 \leq I_{S(50)} \leq 7{,}32$ MPa
- Ultrasound tests: $5910 \leq V_p \leq 7280$ m/s (S2 and S4); $6220 \leq V_p \leq 6440$ m/s (S6)
  $3570 \leq V_s \leq 4590$ m/s (S2 and S4); $3600 \leq V_s \leq 3660$ m/s (S6)

The results of the exploration program and of the in situ/laboratory tests allowed a global interpretation represented by a geotechnical section showing the rock mass zoning pattern (Neiva, 1993) and the table in Figure 5, that includes the geotechnical parameter values for each considered zone.

2.3 Detailed mapping and geological interpretation during construction phase

Excavations for the dam's foundation allowed direct access and subsequent geological and geotechnical detailed mapping of the rock mass (Geonorte, 2011). In general, this mapping confirmed the inferred geotechnical zoning pattern (Neiva, 1993) of the design phase, the excavation depths, the main joint sets and the observed and inferred geological faults.

To create a coherent and accurate dam foundation geological model, EDP gathered and interpreted all the relevant information obtained during design and construction phases and elaborated the dam's reference surface through section presented in Figure 5. This geological section illustrates the most significant gouge filled faults (red), the quartz and/or gouge filled faults (cyan) at valley's bottom, the trace of the apparent dip (section's line of intersection) of schistosity (green), the large sub horizontal joints (black), the geotechnical zone limits (brown) and the hidrogeological 1 Lugeon unit curve (dark blue).



GEOTECHNICAL ZONING (1)					
PARAMETER	ZONE I		ZONE II		ZONE III
	Left bank, valley bottom and right bank up to 643 elevation	Right bank above 643 elevation	Left bank, valley bottom and right bank up to 643 elevation	Right bank above 643 elevation	Left bank, valley bottom and right bank
WEATHERING (W)	W4-W3	W4-W3	W3-W2	W3-W2	W2-W1
FRACTURING (F)	F4-F3	F5-F4	F3-F2	F4-F3	F3-F1
PERMEABILITY (Lugeon Units)	> 10	> 10	> 2	> 2	< 5
COMPRESSION SEISMIC WAVE PROPAGATION VELOCITY (10^3 m/s)	< 4	< 3	4 – 4,5	3 – 4,5	> 4,5
AVERAGE DILATOMETRIC MODULUS (GPa)	5	< 2	6	4	> 10

(1) Adapted from J.M. Cotelo Neiva (1993)

Figure 5 – Geological and geotechnical section through dam´s reference surface.

The geological foundation mapping brought a new and more detailed insight on the foundation geotechnical characteristics and geological structures, particularly on the sub horizontal fault family, first observed at the left bank and, as excavations progressed, also at the right bank and abutment.

These sub horizontal faults filled with clayey gouge of centimetric thickness (< 10 cm in general) were undetected in trench mapping and borehole logging during the exploration program because of the infilling's small thickness, its large spacing and lack of any associated deformation evidences in the rock mass or on other geological structures.

The unfavorable geometry and clayey gouge infilling's low shear strength of these faults led to the analysis and foundation redesign described below, due to its possible influence on dam´s stability conditions, particularly at the intersection of sub horizontal and sub vertical geological faults that could, eventually, allow the formation of sliding wedges.

3 The main aspects of the dam design

The new Alto Ceira Dam was designed by EDP's Department of Dams (EDP, 2007). The adopted shapes of the arch dam were based on parabolic arches used since some decades in the design of several arch dams.

According to the Portuguese Regulations for Safety of Dams (RSB, 2007), and more specifically to the Portuguese Dam Design Recommendations (NPB, 1993), two types of scenarios have to be considered when checking the dam structural safety: exploitation and failure scenarios.

The dam must be able to withstand exploitation scenarios without (or with minor) damage. On the other hand, failure scenarios deal with extreme actions that may lead to important damages in the dam, although overall dam stability must be assured while avoiding uncontrolled reservoir water loss.

For the former scenarios, the relevant load combinations comprised: the dead weight of the independent dam blocks along construction, the hydrostatic pressure at the upstream face for characteristic reservoir exploration levels; the extreme seasonal thermal differences occurring in the structure, with respect to the reference thermal field occurring on the grouting of the contraction joints instant; and the basis design earthquake (BDE). The concrete and foundation mass were considered as isotropic, linear elastic materials with the values E = 20 GPa, ν = 0.2 and α = 10-5 $^{\circ}C^{-1}$ for the mechanical properties.

A detailed analysis of the dam-foundation structural behaviour was supported by 3D finite element models using Ansys software. Limit values of 5.5 and 1.0 MPa for the compression and tensile stresses, respectively, were considered in the following load scenarios: construction phase and reservoir filled up to the normal water level, with and without winter thermal action. The corresponding limit values for the empty reservoir in summer period scenario were 7.0 and 1.5 MPa. The limit values of 0.80 and 0.95 were adopted for the ratio between the tangential and normal forces at the base of each dam block for the normal water level filled up reservoir scenario, respectively with and without winter thermal action, and taking into account the uplift pressures.

The failure scenarios were analyzed in specific studies conducted by LNEC, namely:

- The concrete arch failure under a concrete deterioration scenario;
- The dam analysis for the maximum design earthquake action;
- The foundation failure scenarios, considering the rupture along the dam foundation surface or along discontinuities in the dam foundation.

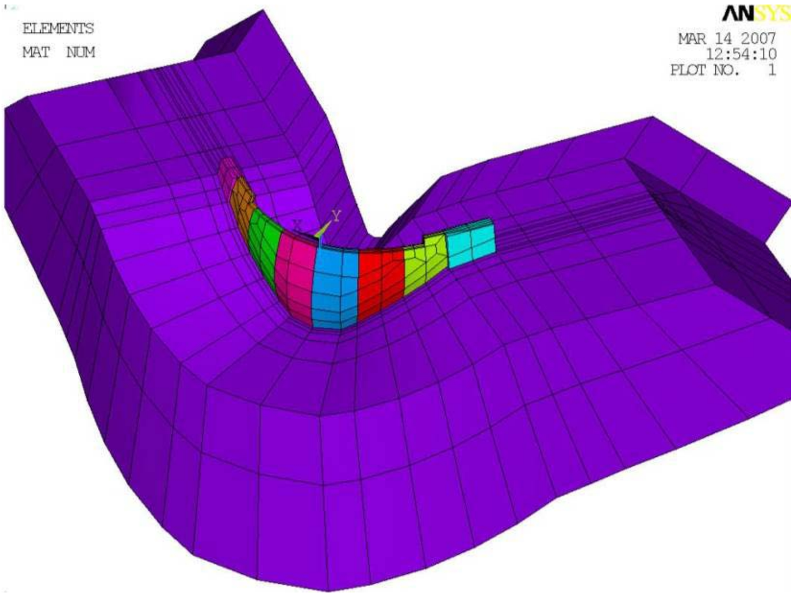


Figure 6 – 3D Finite Element model for exploitation scenarios analysis

The dam foundation safety evaluation (foundation failure scenarios) along weakness surfaces was analyzed by discrete element methods using 3DEC software to model the dam and rock foundation as a system of deformable blocks (LNEC, 2008). Besides the rock mass faults and joints, the contraction joints in the dam body and the concrete-rock interface were also included in the analysis.

The structural analysis performed by EDP for the exploitation scenarios and the analysis performed by LNEC for the failure scenarios demonstrated a good structural behavior of the dam, in accordance with Portuguese Regulations.

4 The redesign of the right abutment

In the dam construction phase, as referred in item 2.3, two important sub horizontal faults with clayey gouge of centimetric thickness were detected, one located in the middle level of the dam foundation in the left bank and the other about 3 meters below the right abutment, parallel to its foundation surface, raised concerns about the dam´s stability.

This situation led to the following conclusions:

- The foundation failure scenarios should be re-evaluated considering the recently detected faults;
- The surface foundation level of the right abutment should be located below the newly detected fault surface in the right bank.

These two actions were immediately implemented. The first one consisted of a complementary study (LNEC, 2011) which is further presented in item 5. For the second one, excavation works were retaken. Figures 7 and 8 show the fault configuration on the right abutment foundation after the complementary excavation was finished. The complementary excavation comprised a volume of 1000 m^3.

Figure 7 - Right abutment after complementary excavation

Figure 8 - Detail of discontinuities in the right abutment foundation

Figure 9 shows the adopted solution for the redesign of the right abutment foundation, which also affected the adjacent arch dam zone block´s foundation surface.

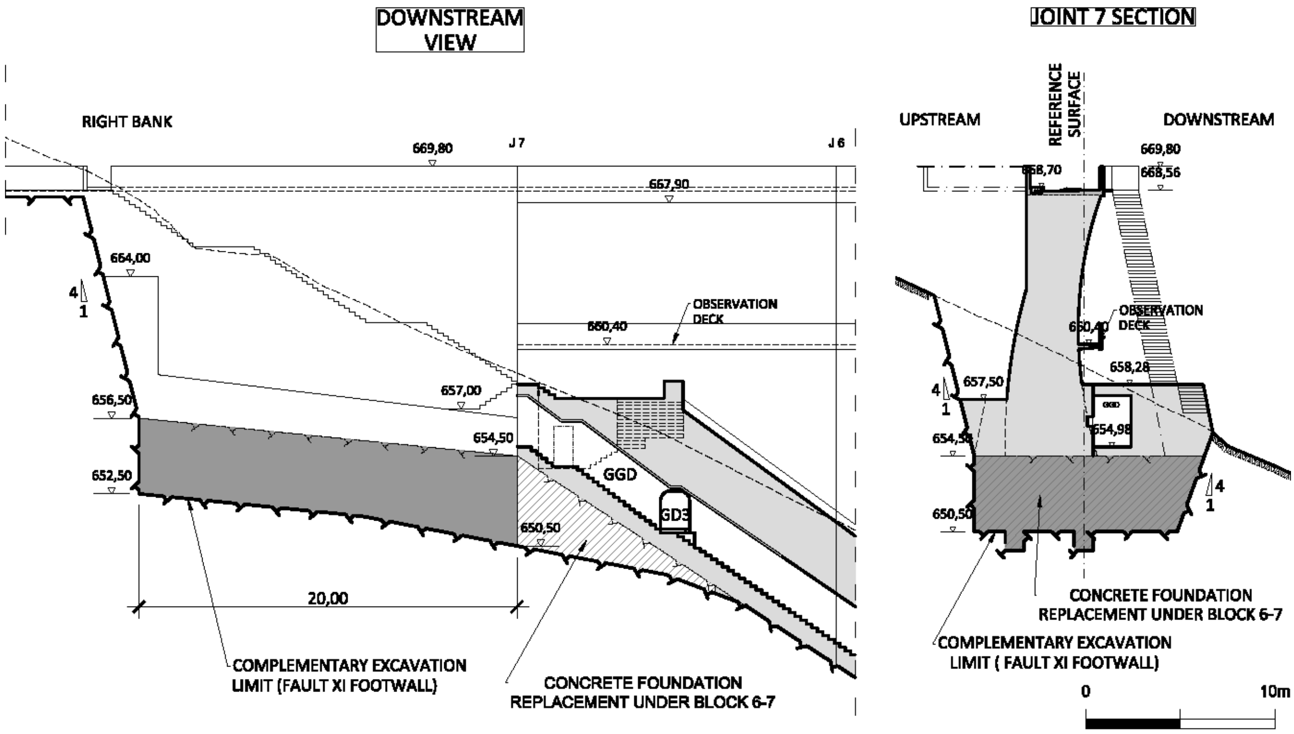


Figure 9 – Right abutment downstream view and section through contraction joint J7

According to the performed analysis, this measure proved to be well appropriate to fulfil the structural safety requirements of the abutment.

5 The reanalysis of the foundation failure scenarios

5.1 Numerical Model

The 3DEC (3-Dimensional Distinct Element Code, Itasca, 2006) was used to simulate the potential failure mechanisms defined by natural rock discontinuities and the concrete-rock interface in order to evaluate the safety and effectiveness of the proposed foundation design.

5.2 Model Generation

The model generation followed the same process applied to the initial analysis of the New Alto Ceira Dam foundation failure scenarios (Lemos et al., 2011). The first step in building the 3DEC model was to import the finite element mesh of the arch dam. Following, blocks below the dam were automatically generated and extended upstream and downstream assuming a cylindrical valley shape. Lastly, larger blocks were created below and laterally

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to obtain the desired model dimensions. To simulate the replacement of rock mass foundation by concrete, a new region located between the dam-rock interface and the sub-horizontal fault on the right-bank was defined as shown in Figure 10.

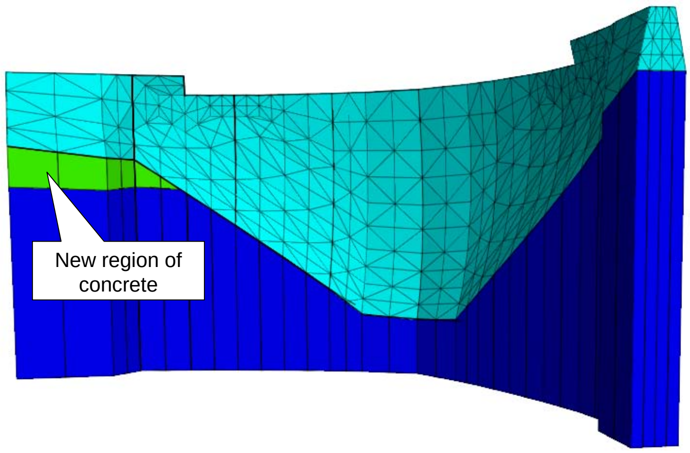

Figure 10 – Definition of the foundation region to be replaced by concrete

During the rock discontinuities modeling process, this new zone was kept uncut in order to represent its monolithic behaviour.

A vertical discontinuity was placed in the upstream edge of the concrete-rock interface, which accounts for the expected rock joint opening in this tensional area. As in the original analysis, only the discontinuities located on the right bank were represented in order to reduce the computational size (Figure 11).

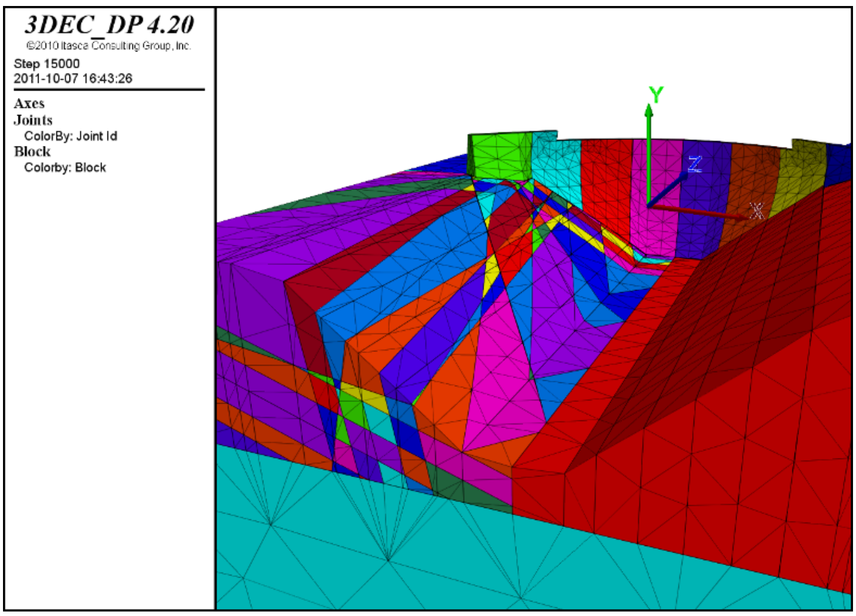

Figure 11 – View of the right bank model

5.3 Material properties

All blocks were considered elastic, with the nonlinear behaviour restricted to the discontinuities. It should be remarked that, even if this was a discontinuous model, the number of rock joint planes was quite small. Therefore the Young's modulus of the rock blocks needed to be selected to provide the global deformability of the jointed rock. The values of 20 GPa for the Young's modulus and 0.2 for the Poisson coefficient were considered for both concrete and rock materials.

The Mohr-Coulomb model was adopted for the discontinuities behaviour. All rock discontinuities were assumed to have only frictional strength, without dilatation. Dam contraction joints were also taken as purely frictional. The dam foundation surface was assigned cohesive and tensile strength. Table 1, shows the adopted properties for the discontinuities.

Table 1- Mechanical properties of the discontinuities

Properties	Unit	Faults	Joints	Upstream vertical discontinuity	Dam-rock interface	Dam contraction joints
Normal stiffness	GPa/m	3,7	3,7	10	10	10
Tangential stiffness	GPa/m	2,6	2,6	5	5	5
Friction angle	°	20	38	38	45	40
Cohesion	MPa	0	0	0	3	0
Tensile strength	MPa	0	0	0	2	0

5.4 Modelling steps

The analysis procedure comprised a sequence of modelling steps, which followed, as much as possible, the physical path. The first step corresponded to the in situ condition, before dam construction. The second modelling step was the simulation of the replacement of the rock-mass foundation by concrete followed by the dam construction, in which gravity is applied to the cantilever blocks. Afterwards, reservoir filling was simulated by applying the hydrostatic pressure to the dam upstream face. Lastly, the water pressures in the discontinuities were introduced.

5.5 Safety evaluation

Safety factors were evaluated by means of a strength reduction procedure. In the rock discontinuities, the friction coefficients were divided by progressively larger factor until collapse took place or displacement magnitudes reached unacceptable levels. In the rock joints and faults, assumed to be purely frictional, the reduction factor was applied to the tangent of their friction angles. For the concrete-rock interface the condition of no cohesion or tensile strength was imposed.

The development of a failure wedge on the right bank, shown in Figure 12, was obtained for a friction reduction factor of 1.7. This failure mechanism concerned only the stability of the downstream rock mass, while the dam foundation remained stable until a reduction factor of 1.9 was applied.

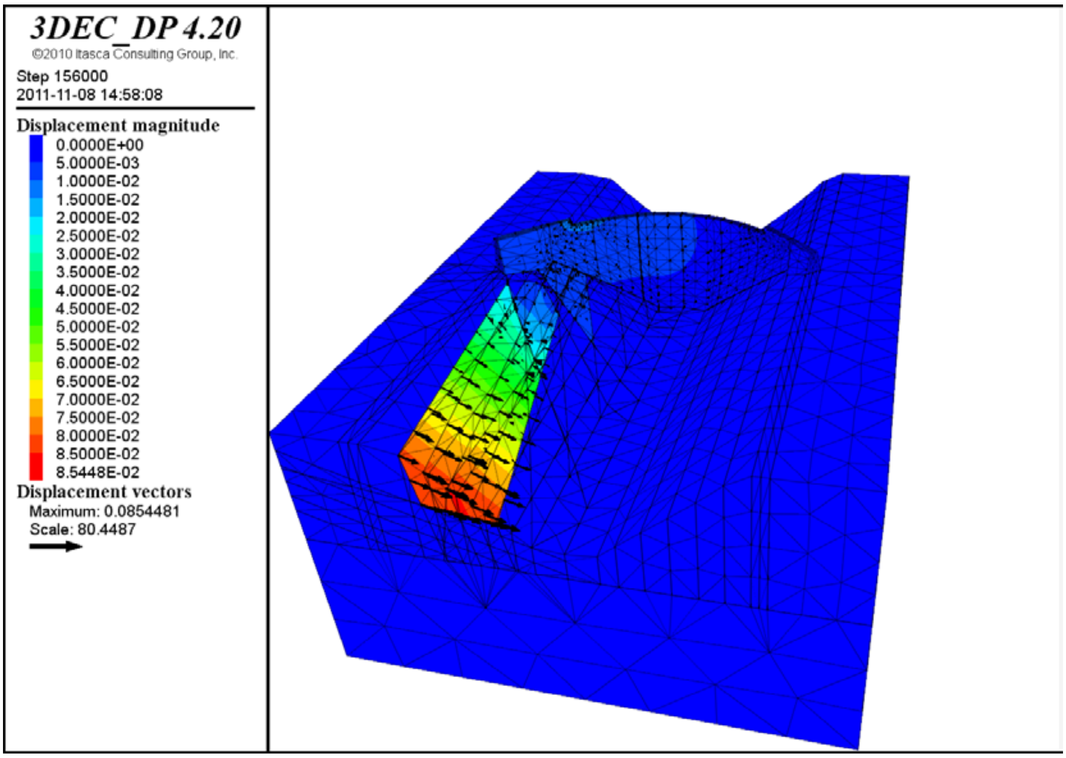

Figure 12- Displacement field for a reduction factor of 1.7

The displacement indicators in Figure 13 (solid lines) show that the joint slip starts to influence dam displacements when the reduction factor exceeds 1.5, and that a clear acceleration can be seen above 1.8. The previous foundation surface shear curve (dashed line) represents the displacement obtained for the model without rock mass foundation replacement by concrete.

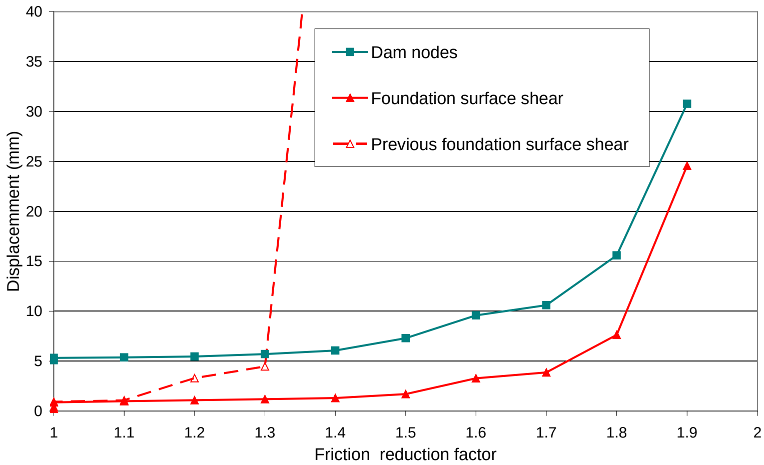

Figure 13 Evolution of model displacements with strength decrease factor

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6 Final considerations

However extensive and thorough, the geological survey program conducted in the New Alto Ceira Dam site left two unpredicted foundation geological accidents to be revealed only after excavation works, in the course of the dam construction phase.

One of the discovered unfavourably oriented and low resistance clay gouge filled fault immediately concerned the design team as it could considerably lessen the dam-foundation structural strength.

Under those circumstances, it was decided to redesign some structural elements and their foundation conditions, so that the same safety criteria of the original design could be achieved.

Modelling of foundation failure scenarios employing discrete element block models has been recurrently implemented by LNEC and EDP for safety evaluation of dams. The versatility of discrete elements allows the consideration of complex rock jointing patterns, general constitutive models for all discontinuities including dam joints and the foundation surface, and of water pressure effects on joints.

Several models with different discontinuity patterns and properties were easily assembled, allowing multiple failure modes to be examined and validated, in compliance with the safety standards.

As concluded for the dam design phase, the safety assessment procedure based on the progressive reduction of joint strength properties provides a helpful insight into the effects of geologic features on structural behaviour. Moreover, the 3DEC model versatility allowed the quick analysis of the abutment's new foundation scenarios, supporting the design team decision process during construction phase when the new foundation conditions and remedial design changes to the dam had to be validated in terms of safety conditions in a short period of time, compatible with the overall construction schedule.

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