Modelling of arch dam foundation failure scenarios - case studies of Baixo Sabor and Alto Ceira dams

J V Lemos[1)] and N S Antunes[2)]

ABSTRACT

In concrete arch dam foundations, potential failure mechanisms are typically defined by natural rock discontinuities or the concrete-rock interface. Discrete element models are particularly adequate for the safety assessment of these failure scenarios, given their ability to represent the geologic structure of the rock mass, as well as the concrete cantilevers. Application of these procedures to two large dams presently under construction in Portugal, i.e. Baixo Sabor and Alto Ceira arch dams, is presented. The numerical models represent the foundation rock mass as a system of deformable blocks defined by the main faults and joint sets identified at the site. The safety analysis is performed by progressive reduction of strength along the discontinuities until the failure condition is reached.

Keywords: Arch dams, rock foundations, failure analysis, discrete elements.

1. INTRODUCTION

In the design of concrete arch dams, the safety of the foundation rock mass has to be thoroughly assessed. The analysis of potential failure of rock blocks defined by the rock mass discontinuities, considering the installed water pressures, is a crucial task in dam design. Simplified analytical and graphical methods were developed for this purpose and successfully applied in the design of many dams[1]. Presently, numerical models are the preferred simulation tool. Finite element analysis provides a more comprehensive representation of the rock mass behaviour, either by means of equivalent continuum idealizations or by employing joint elements[2]. These tools are routinely employed to assess arch dam performance and safety, considering operating and failure scenarios[3]. Discrete element methods are widely used to obtain discontinuum idealizations[4] at various scales, ranging from laboratory test analysis to large rock engineering projects. These numerical techniques treat the rock mass as an assembly of component blocks or particles in

[1)]Laboratório Nacional de Engenharia Civil (LNEC), Av. do Brasil, 101, 1700-066 Lisboa, Portugal. Email: vlemos@lnec.pt, webpage: www.lnec.pt, [2)]EDP Gestão da Produção de Energia, Av. José Malhôa, Lt. A13, 1070-157 Lisboa, Portugal. Email: nuno.antunes@edp.pt, webpage: www.edp.pt

mechanical interaction, being particularly suitable for the evaluation of failure modes, either under static or dynamic conditions, allowing the simulation to progress into the large displacement range. They are also applicable to arch dam analyses[5], as they allow the representation of the geologic structure of the rock mass, as well as the concrete vault with its contraction joints and foundation surface.

In this paper the application of discrete element models to the safety evaluation of arch dam foundations is discussed, focusing on the case studies of two large dams presently under construction in Portugal, i.e. Baixo Sabor[6] and Alto Ceira[7] arch dams. In the numerical models developed, the concrete arch is composed of elastic cantilevers separated by joints with no-tension behaviour. The model also includes the dam-foundation surface and the relevant rock mass discontinuities, including faults with known location and joints from the major sets identified at the site. The effects of dead weight, reservoir water pressure, uplift pressures and water pressures in rock joints were taken into account.

A safety evaluation procedure, based on the minimization of the strength properties of the rock discontinuities, was employed to verify the compliance with regulatory requirements. The analyses were conducted with the discrete element code 3DEC[8], widely used in the field of rock mechanics, which has specific features that make it a versatile tool for dam foundation studies. The main steps of the modelling procedure are described in the next sections, with reference to the two case studies.

2. BAIXO SABOR ARCH DAM

2.1 Arch dam

The hydroelectric project of Baixo Sabor is located in the northeast of Portugal in the lower branch of the Sabor River, a tributary of the right bank of the Douro River. The scheme is composed of two dams, a 123m high arch dam upstream, and a 45m high gravity dam downstream. Both power houses will have reversible units to enable pumping from the Douro River to the large reservoir created by the upstream dam. The dam construction is presently under way.

The arch dam[6] (see Figure 1) has a crest length of 505m and a total concrete volume of 670,000m^3. The full storage level is at elevation 234 and the reservoir capacity is 1095Mm^3.

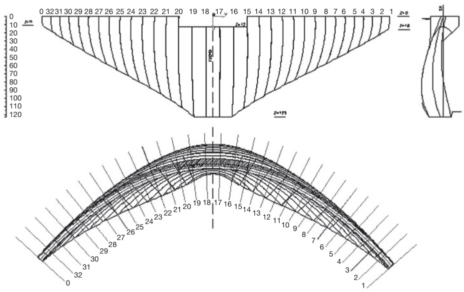


Figure 1. Baixo Sabor arch dam.
Elevation, cross-section of central cantilever, and plan view

2.2 Rock mass characterization

At the dam site, the river valley follows an essentially straight course for about 1km, with the valley bottom, 40m wide, at an elevation of approximately 127. The mean slope of the banks is about 40°. The dam is to be founded on a sound granitic rock mass[9]. At the site four joint sets were identified, the three main ones being orthogonal: the main set is approximately sub-vertical and parallel to the river; the second set is sub-horizontal, dipping slightly upstream; and the third is sub-vertical, normal to the valley axis. Figure 2 shows the main faults detected in the vicinity of the dam foundation. An extensive programme of rock mechanics testing, comprising *in situ* and laboratory tests, was undertaken by LNEC[10].

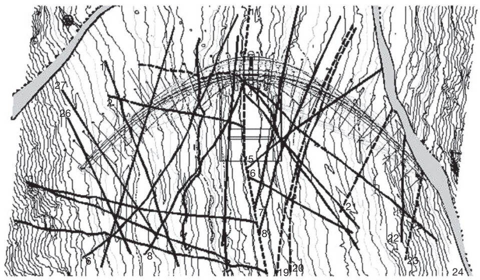


Figure 2. Location of main faults and dykes

2.3 Generation of the numerical model

The code 3DEC[8] has special features that simplify the analysis of arch dam foundations. As in most applications, the rock mass is represented by deformable blocks of polyhedral shape, which are internally divided into a finite element mesh of tetrahedral uniform strain elements. For the concrete body of the dam, however, the code allows the use of 20-node solid brick elements, much more efficient for the accurate representation of the bending behaviour. The finite element mesh is generated outside and then imported into 3DEC. In this case, the mesh has two elements across thickness, and the vertical contraction joints between the cantilevers were considered.

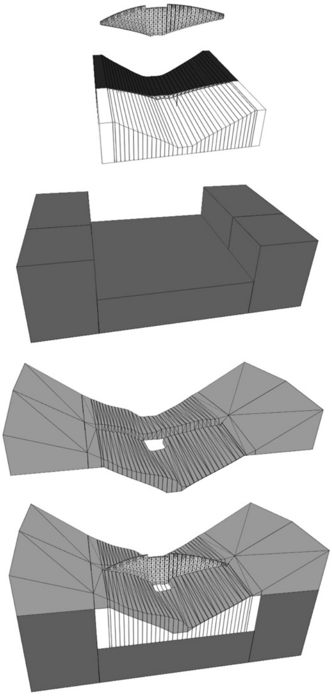


Figure 3. Geometric components of 3DEC dam-foundation model

Figure 3 illustrates the process of building the geometric model. At the top, the arch FE mesh is shown. Blocks below the dam are automatically generated, and extended upstream and downstream, assuming a cylindrical valley shape. Larger blocks are created below and laterally to obtain the desired model dimensions. Often, the global geometry of the model obtained at this stage is used in the analysis for simplicity, i.e. keeping the cylindrical valley shape. In the present case, however, the actual terrain topography was represented by inserting a layer of blocks above, as shown in this figure.

The picture at the bottom of Figure 3 thus represents the global model geometry. The lines delimiting the foundation blocks are simply a means of defining the shape of the model components in terms of a set of convex blocks, and they are not true discontinuities. These joints play no role in the analysis as the blocks are effectively joined by constraining the adjacent nodes to move together. At this stage, therefore, the rock mass joints are not yet included, the only discontinuities being the vertical contraction joints in the arch and the concrete-rock interface. After generating a mesh of tetrahedral elements in the rock blocks, performed automatically in 3DEC, this model may be used to analyze a scenario of failure along the foundation surface, assuming the rock mass to be an elastic continuum.

The next step in the model generation regards the main faults identified at the site. The location of these major features is known, and they can be inserted in the model by cutting the rock blocks. Then the joints sets are represented by selecting a few joints of each in order to create blocks that may be part of kinematically unstable regions. To keep the computational costs reasonable, only a limited number of joint planes are usually selected, which requires careful judgement. In fact, it is often better to create more than one model, choosing different planes of each set, rather than trying to include every feature in a single analysis, which may substantially increase run times, as well as the time to prepare and verify the data, and to interpret the results.

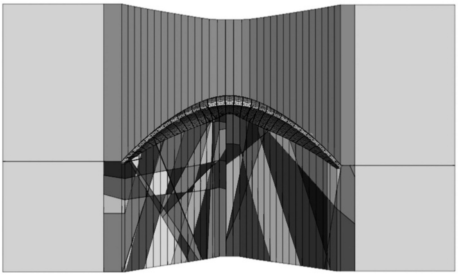


Figure 4. Top view of model with rock discontinuities (hiding the upper terrain block layer)

Figure 4 shows a top view of the model with the discontinuous foundation. The top layer of blocks, representing the topography, was hidden for clarity. The main faults identified in Figure 2 are included, as well as a few joint planes of the sub-horizontal set in the vicinity of the dam foundation surface.

Figure 5 shows a perspective of the complete model, including the surface topography. It may be seen that the faults and rock joints were not extended upstream. This is a conservative simplification, which prevents the upstream rock mass from constraining the failure modes. A vertical discontinuity is placed along the upstream edge of the concrete-rock interface, which accounts for the expected rock joint opening in this tensioned area. In the case of this dam, a sub-vertical joint set normal to the river axis is actually present, so this simplifying assumption is entirely justified. This vertical joint is assigned rock joint properties, without tensile strength, and water pressures given by the full reservoir conditions are applied.

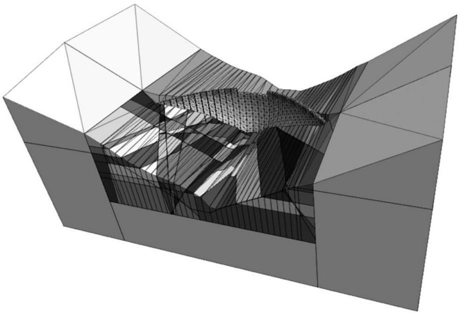


Figure 5. Model with dam and main rock discontinuities

The full model comprises about 2300 blocks, which once discretized have about 26,100 grid-points, thus leading to a total of 78,300 degree-of-freedom. In these figures the construction lines necessary to define the geometry are still visible.

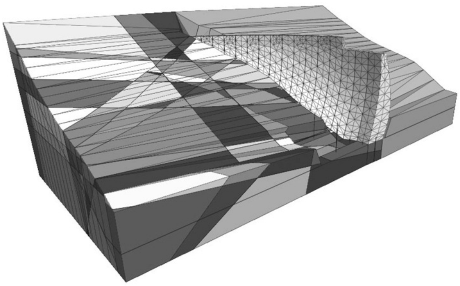


Figure 6. View of the right bank half of the global model

Figures 6 and 7 show separate views of the right bank and left bank halves of the model. In these pictures, the upstream slope of the sub-horizontal joints, which reduces the propensity to sliding on this set, is clearly visible.

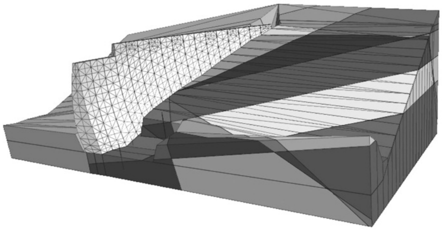


Figure 7. View of the left bank half of the global model

It should be noted that in this model all the rock joints are represented by through-going planar cuts, implying trace lengths clearly larger than the ones to be expected, given the *in situ* data collected. This is clearly a conservative assumption, as it neglects joint non-planarity and rock bridges, which would prevent or restrain extensive sliding. In some cases, however, this simplification may be excessively conservative, and more realistic joint trace lengths may have to be considered. In such models, with non-persistent joints, the possible failure of rock bridges needs to be considered.

2.4 Modelling steps

The analysis procedure comprises a sequence of modelling steps. Once the model geometry is created, as described in the previous section, mesh generation is performed for all the rock blocks. Material properties must then be assigned to concrete and rock blocks and joint properties prescribed for all discontinuities, a task that needs to be carefully verified for systems with complex joint patterns.

The first step corresponds to the *in situ* condition before dam construction. In this case, *in situ* stress measurement results were available from geotechnical investigations conducted for the power house design, which is located near the dam. A stress state based on this data was initialized in the rock blocks, even if this is not usually a critical parameter for dam failure modes. The dam construction step follows, in which gravity is applied to the cantilever blocks. Afterwards, reservoir filling is simulated by applying the hydrostatic pressure to the dam upstream face.

The water pressures in the discontinuities, corresponding to the filling of the reservoir, were then introduced. Along the concrete-rock interface, uplift pressures were prescribed according to the usual design criterion, a bi-linear diagram with 1/3 of the reservoir head at the drain location. In the rock discontinuities the full reservoir head was considered upstream, whereas downstream a simplified pressure field was prescribed, defined in terms of a water table compatible with the valley slopes. The current 3DEC code actually allows an analysis of fluid flow in the rock joints to be performed, but there is often not enough information to undertake such studies at the design stage.

In addition, a fracture flow analysis requires a network with many more joint planes than those necessary for the current failure analysis. A more practical alternative is to perform an equivalent continuum flow analysis to calculate water pressures that are then applied in the joints of the block model used in the mechanical failure study. 3DEC may also be employed in such continuum flow analysis[11].

Safety factors were evaluated by means of a strength reduction procedure. In the rock discontinuities, the friction coefficients were divided by progressively larger factors until collapse took place, or displacement magnitudes reached unacceptable levels.

2.5 Material properties and main results

As already stated, all blocks were considered elastic, with non-linear behaviour restricted to the discontinuities. It should be noted that even if this is a discontinuum model, the number of rock joint planes is quite small, so the Young's modulus of the rock blocks needs to be selected to provide global deformability of the jointed rock. In this case, a value of 20GPa was considered, following the results of large flat jack and dilatometer tests[10].

Laboratory tests of the rock joints[10] provided values of joint normal and shear stiffness. In the numerical models constant stiffness values were used, considering the range of normal stresses expected. The friction angles of the three main sets had average values of between 37° and 39°. All rock discontinuities were assumed to have only frictional strength, without dilation. Dam contraction joints were also taken as purely frictional, thus neglecting the contribution of shear keys.

The dam foundation surface was assigned cohesive and tensile strengths in most analyses. The adopted joint constitutive model adopted assumes brittle behaviour, reverting to frictional strength in case of tensile or shear failure. However, it is common to also consider a scenario of failure through this surface only, in this case assumed to resist only by friction. In this analysis the model depicted in Figure 3, i.e. before inserting the rock discontinuities, was employed. The results showed that collapse took place for a frictional angle of 32°, which corresponds to a safety factor of 1.6 if we admit a friction angle of 45° and neglect the contribution of cohesion.

For the study of failure modes through the rock discontinuities, besides the model shown in Figures 5-7, two other models were built. Each one of these new models focused on one of the abutments, introducing only the rock discontinuities on half of the model. In this way, the failure modes of the right and left banks were analyzed independently, which also allowed more joint planes to be introduced without excessive computational cost.

The conclusions obtained from the three alternative representations of rock mass jointing were essentially similar. Substantial displacement only took place when the tangent of the friction angle of the rock joints and faults was reduced by a factor of 2, which confirms the large safety margin expected, given the joint orientation. Figure 8 shows in detail the displacement field on a horizontal cross-section, where the initiation of the movement of a wedge under the right abutment is visible, triggered by the friction decrease.

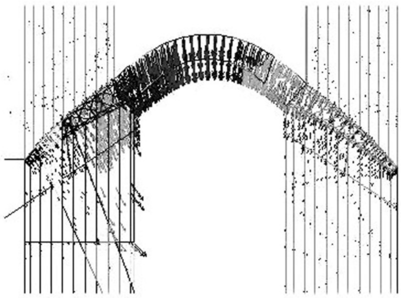


Figure 8. Pattern of displacements for developing failure mode (horizontal cross-section)

3. THE NEW ALTO CEIRA ARCH DAM

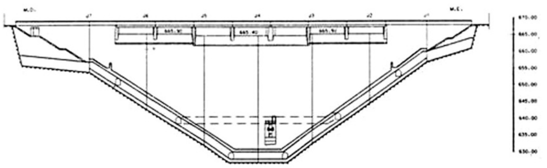
3.1 Dam features

The new Alto Ceira dam[7] currently under construction on the Ceira River in the centre of Portugal was designed by EDP to replace the existing arch dam which had been damaged by alkali aggregate reactions (AAR). The new dam, shown in Figure 9, is located 200m downstream of the old one. It is a double curvature arch, 41m high, with a crest length of 100m, and is supported by two gravity abutment blocks. The dam is composed of six blocks, 16-17m wide, separated by vertical contraction joints. The central cantilever has a thickness varying from 2m at the crest to 5.5m at the base. The total volume of the dam concrete is 17,000m^3.

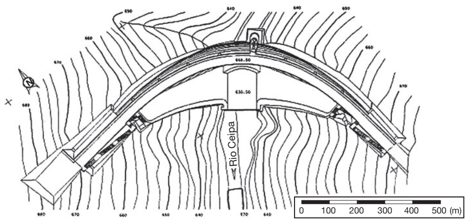
3.2 Rock mass representation

The assessment of the potential failure modes involving the rock foundation was performed by means of a 3DEC model, following the methodology presented in the previous section.

The rock mass is schistous, displaying some areas of significant weathering at the top of the slopes. The schistosity is sub-vertical, dipping upstream, and approximately normal to the valley axis. Several joint sets, mostly sub-horizontal or sub-vertical, were identified at the site.



(a)



(b)

Figure 9. New Alto Ceira dam. Elevation and plan

The first step in building the 3DEC model was to import a finite element mesh of the arch, and then to generate the valley shape, as previously explained. From the various known faults only two were selected for representation in the 3DEC model (Figure 10).

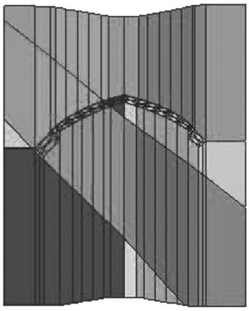
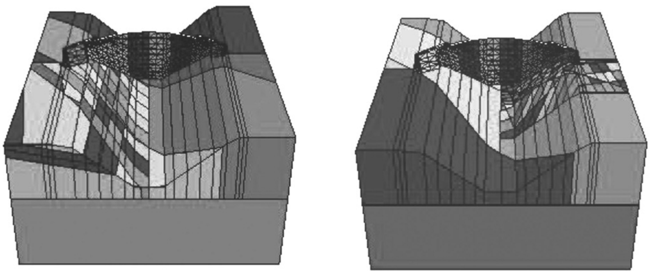


Figure 10. Plan view of numerical model with dam foundation surface and fault traces

Given the complexity of the jointing and the differences of orientation in each valley side, it was decided to build separate models for each bank. Furthermore, 3 different models were built for the right bank, each one combining 2 joint sets capable of forming potential failure wedges under the dam. Figure 11a shows one of these models. Figure 11b displays the model of the left bank, in which 3 joint sets were considered. As the critical failure wedges were in the vicinity of the dam, joints were only inserted in a limited region, in order to reduce the computational size.



(a) (b)

Figure 11. View of one of the right bank models (a), and the left bank model (b)

As in the Baixo Sabor models, the dam contraction joints and the foundation surface are also represented, as well as a vertical joint at the upstream edge of the foundation surface, allowing the tensile separation from the upstream rock mass.

3.3 Safety evaluation results

The safety evaluation methodology was similar to the one described for the Baixo Sabor dam in Section 2.4. As before water pressures were applied at the concrete-rock interface and in all rock discontinuities. Rock joint laboratory tests indicated friction angles of 33° for the joints associated with the sub-vertical schistosity, which play no role in the failure mechanisms, and 38° for the other sets. Progressive reduction of the friction coefficient allowed the development of the failure modes. Figure 12 displays the results obtained in the right bank model of Figure 11a. The evolution of three displacement indicators is plotted against the strength reduction factor (F): the top curve corresponds to the maximum nodal displacements in the upstream-downstream direction, reflecting the dam crest movements; the two other curves indicate maximum shear at the foundation joint and at the rock mass joints. It can be seen that an increase of movement occurs when F reaches 1.4, with a jump at $F = 1.8$, when sliding starts to spread along the foundation surface. In the next step ($F = 1.9$), substantial sliding took place, indicating impending structural collapse. The results obtained in the various numerical simulations demonstrated an adequate safety margin against failure modes through the foundation.

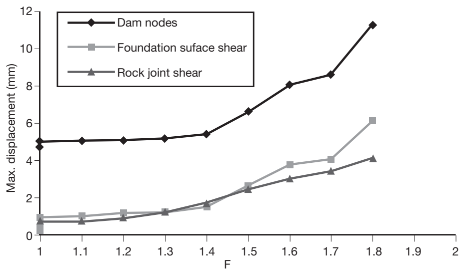


Figure 12. Evolution of model displacements with strength reduction factor

4. CONCLUSIONS

Modelling of foundation failure scenarios for arch dams has been addressed by means of discrete element block models. For dam foundation studies, it is important to use deformable rock blocks, and the concrete structure behaviour also needs to be accurately represented. The 3DEC models employed resort to higher order elements for the dam mesh and tetrahedral elements for the rock blocks.

The versatility of discrete elements allows the consideration of complex rock jointing patterns, general constitutive models for all discontinuities, including dam joints and the concrete-rock interface surface, as well as the effects of joint water pressures.

The analysis performed for Baixo Sabor and Alto Ceira dams showed that a safety assessment procedure, based on the progressive reduction of joint strength properties, provides a helpful insight into the effects of geological features on structural behaviour. In both cases, several models with different discontinuity patterns and properties were easily assembled, allowing multiple failure modes to be examined, and the verification of the compliance with safety standards.

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