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Discontinuum models for dam foundation failure analysis

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ABSTRACT: The safety assessment of concrete dam foundations entails the examination of the potential failure mechanisms, typically defined by natural rock discontinuities or the concrete-rock interface. Numerical models which represent the rock mass as a discontinuous medium, in particular discrete element models, are particularly adequate for the analysis of these failure scenarios, given their ability to represent the geologic structure of the rock mass, as well as the concrete structure. The application of discrete element deformable block models to the safety assessment of arch and gravity dam foundations is discussed with reference to specific examples. The issues involved in model generation, such as the representation of the rock discontinuities or the application of joint water pressures, are examined, as well as the procedures for safety factor evaluation.

Subject: Modeling and numerical methods

Keywords: Rock slopes and foundations, Stability analysis, Numerical modeling

1 INTRODUCTION

The design of new concrete dams, or the safety assessment of existing dams, requires particular attention to be paid to the behavior of the foundation rock mass. Surveys promoted by ICOLD and other institutions have shown that many of the incidents or deficiencies experienced by concrete dams are linked to the foundation. The accident of the Malpasset arch dam is the best known example of a structural collapse caused by sliding on rock discontinuities (e.g. Londe 1987). In fact, this event motivated a substantial body of research on rock foundations issues, namely on the hydro-mechanical behavior, and stimulated the development of new methods of safety assessment. Analytical or graphical methods, such as those proposed by Londe (1973), became standard tools in arch dam design. With these techniques, it became possible to analyze the potential failure of rock blocks defined by the rock mass discontinuities, considering the installed water pressures and the dam loads. More recently, Goodman & Powell (2003) applied Shi and Goodman's Block Theory to identify moveable blocks in concrete dam foundations. All these techniques, based on simple block mechanics, remained important for safety evaluation, while the finite element models became the preferred tools to analyze dam foundations under operating conditions and to predict stresses and displacements (e.g. Wittke 1990).

Discrete element (DE) models are widely used in rock mechanics. As will be discussed in the following section, they can be used either at the micro or meso-mechanical scale, e.g. to investigate fracture phenomena, or at the engineering scale, e.g. to analyze a full arch dam foundation. In this paper, the latter perspective is adopted. In its simplest form, a DE rigid block model may be viewed as a numerical tool that performs the same stability analysis as Londe's method. However, not just statics are involved, but a full mechanical analysis is undertaken. A DE deformable block model, with internal meshes in the blocks, is capable of stress and displacement analysis as a finite element model, while retaining the ability to simulate in a straightforward manner failure modes defined by the rock discontinuities. This is the type of model that will be examined in this paper as an engineering tool for dam foundation failure analysis. It is not intended to represent in detail the rock mass jointing, but instead employs a relatively coarse block structure effectively directed at the assessment of safety with respect to specific deformation and collapse modes.

The paper outline is the following. The essential concepts of discontinuum modeling are briefly addressed in the next section. Then, the main issues involved in the application of DE block models to arch dam foundations are examined, namely model generation, representation of jointing, water pressure fields, and safety factor calculations. The discussion of these items will be illustrated by several examples of dam studies performed with the code 3DEC (Itasca 2006). Finally, some topics related to gravity dam failure analysis will be addressed.

2 DISCONTINUUM MODELING

An engineering model is necessarily a simplification of the physical reality, more often intended to answer a specific question, e.g. about safety or performance of a proposed design or an existing structure, rather than to provide a meticulous description of nature. The amount of detail to be included in the model is dictated by the purpose of the analysis, e.g. stability assessment or interpretation of monitoring data in operating conditions, and is always limited by the experimental data available.

Two fundamental options exist for the representation of a jointed rock mass: (i) the equivalent continuum approach, in which a continuum constitutive model is employed to represent in an average manner the effects of the discontinuities; (ii) the discontinuum approach, in which the discontinuities are explicitly represented individually. Both of these idealizations have their fields of application, and often a combination of the two is advisable: a number of key discontinuities are modeled explicitly, while the others are lumped into the block behavior.

The finite element method is the most common tool for equivalent continuum analysis, but it is also capable of addressing discontinuous models by means of joint, interface

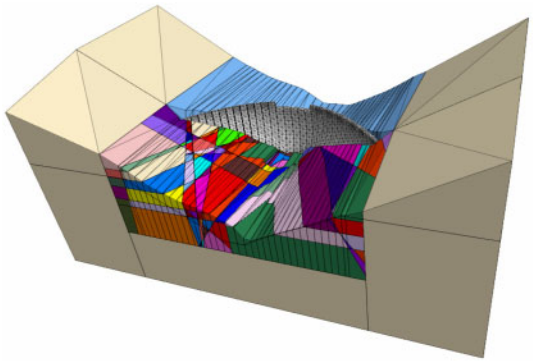

Figure 1. DE block model of Baixo Sabor arch dam.

or gap elements (e.g. Alonso et al. 1994). The development of the distinct element method by Cundall in 1971 was aimed at a direct representation of a discontinuum. Presently, the designation of "discrete elements" (DE) covers a wide family of numerical methods (distinct elements, discrete finite elements, DDA, etc.), all sharing the concept of representing a discontinuous medium as an assembly of blocks or particles. These idealizations are applicable at various scales of analysis. At one end, we have the micromechanical models based on many rigid particles or blocks employed, for example, in fracture analysis at lab test scale. At the other end, the deformable block models of large civil and mining engineering works. Although the increase of computer power is expanding the range of the micro-models (Cundall 2001), it is the latter type of DE model that is most suitable for dam foundation analysis (Fig. 1).

The analysis of collapse mechanisms in dam foundations involves the representation of the discontinuities where sliding may take place. In this field of application, the block structure thus defined is better replicated in a numerical model by means of deformable blocks. In this way, a more realistic simulation of the distribution of structural loads is obtained, influenced by the foundation properties and their spatial variation, even with a fairly coarse block system. In the code 3DEC (Itasca 2006), deformable blocks are obtained by internal discretization into a finite element mesh of tetrahedra. For dam foundation studies, these rock blocks are typically assumed elastic, with all the nonlinear behavior concentrated on the joints. For arch dams, the correct bending behavior is more easily achieved with higher order elements, thus 3DEC allows 20-node bricks to be used for the concrete vault. In this type of model, the vertical contraction joints and the concrete-rock interface are also discontinuities which may be assigned general constitutive models. The Mohr-Coulomb model is the most widely used, but many other rock joint models exist.

The variety of numerical techniques presently available often brings the question of what are their real differences, e.g. how does a DE deformable block model differs from a FE model with joint elements. If both models share the constitutive assumptions regarding block material and joint behavior, then their response should be not be dissimilar. It is mainly the numerical approach that sets them apart. FE models represent block interaction by means of joint elements, while DE models typically use point contacts. FE packages favor matrix and implicit solvers, while DE codes obtain static solutions by dynamic relaxation. The same explicit algorithm, but with real values of damping, is employed in time domain dynamic analysis. Most standard FE analyses assume geometric linearity, while DE codes are designed to extend the solution into the large displacement range, with automatic update of block connectivity.

Safety requirements for dams cover both operating and extreme conditions (e.g. Pedro 1995). The monitoring of concrete dam foundations over the years has produced extensive databases to validate and calibrate numerical models for conditions of normal operation. It is much more difficult to be confident about the models' ability to evaluate failure scenarios. Back analysis of accidents is an important test, but laboratory experiments with physical models remain a valuable source of information (e.g. Gomes 2006, Fei et al. 2009). Benchmark comparisons between different codes are also very helpful, namely to examine numerical and implementation issues.

3 ARCH DAM FOUNDATION ANALYSIS

3.1 *The case of Baixo Sabor dam*

The main topics involved in the application of DE models to the analysis of failure mechanisms in arch dam foundations will be discussed resorting to a few examples, in particular to Baixo Sabor dam, presently under construction. The Hydroelectric Project of Baixo Sabor, owned by EDP, is located in the north-east of Portugal in the lower branch of the Sabor river, a tributary of the right bank of the Douro river. It is composed of two dams, a 123 m high arch dam upstream, and a 45m high gravity dam downstream. Both powerhouses will have reversible units to enable pumping from the Douro river to the reservoir created by the upstream dam with a capacity of 1.1×10^6 m^3. The design was performed by EDP (Matos et al. 2007). The arch dam has a crest length of 505 m and a total concrete volume of 670000 m3. The 3DEC model for foundation safety assessment, shown in Figure 1, is described in more detail by Lemos & Antunes (2011). The full model comprises about 2300 deformable blocks with 26100 grid-points (the internal block mesh is not shown in the figure).

3.2 *Model generation*

Various techniques and software are now available for the generation of numerical models, with most analysis codes capable of using a combination of built-in tools and data input from external geometric modelers. For dam foundation analysis with the code 3DEC, the model generation typically starts by reading a FE mesh of the dam. The mesh of the Baixo Sabor dam model is plotted in Figure 2, with the contraction joints separating the structure into the cantilever blocks. This mesh consists of higher order brick elements (in the plot the faces are triangulated). More than one element across the thickness is normally recommended to provide a finer discretization of the foundation surface. Block interaction mechanics in most DE codes is based on sets of point contacts, rather than interface elements, which allows the contact between blocks with unmatched meshes, as happens in the present case along the foundation surface (e.g. Lemos 2008).

The global model geometry may be created, in the simplest case, by extending the model upstream and downstream from the dam-rock interface, assuming a cylindrical valley shape, as shown at the top of Figure 3. Alternatively, the actual surface topography may be input, as a new layer of blocks placed above, as shown at the bottom of the same figure. The model in the top figure may be used for the analysis of sliding mechanisms on the dam-rock interface, in the simplified scenario of

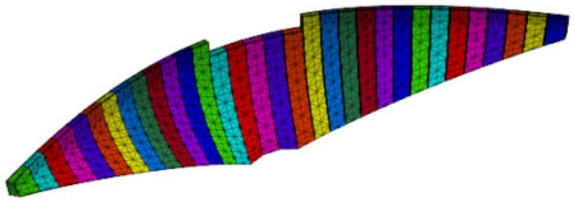

Figure 2. FE mesh of concrete arch.

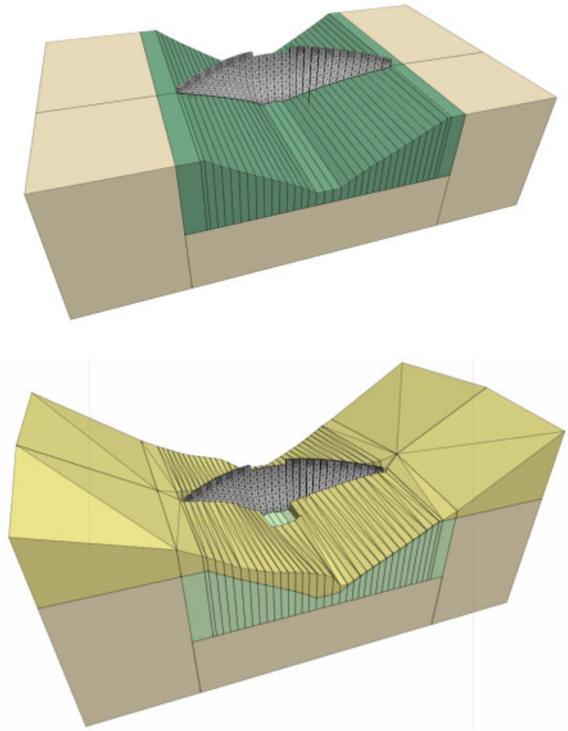

Figure 3. Model geometry: simplified valley shape (top) and model with terrain topography (bottom).

a continuous foundation, but considering the cantilever block structure of Figure 2.

3.3 *Representation of rock mass discontinuities*

3.3.1 *Selection of discontinuities*

The models in Figure 3 simply define the rock mass geometry in terms of a set of rigidly attached or joined blocks. The next phase in the model generation is the representation of the rock discontinuities. It is at this stage that critical decisions need to be made about what features to include and how to represent them in a necessarily simplified manner. Typically, there are several faults or other major features identified at the site (e.g. Fig. 4), which may be inserted at their known locations with given orientations. Then, each of the most significant joint sets is represented by a few selected joint planes. The purpose of the analysis, stability assessment, directs the selection of the number and location of joints, as the intention is not to recreate in detail the joint structure, but to identify the possible failure modes and their likelihood. In practice, a small number of joints from each set is normally sufficient to define the most relevant mechanisms. Valuable lessons can be drawn from the clear rationale manifest in the classical papers (e.g. Londe 1973).

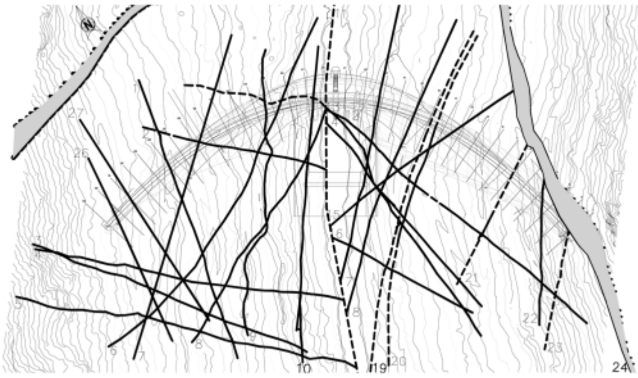

Figure 4. Location of main faults and dykes.

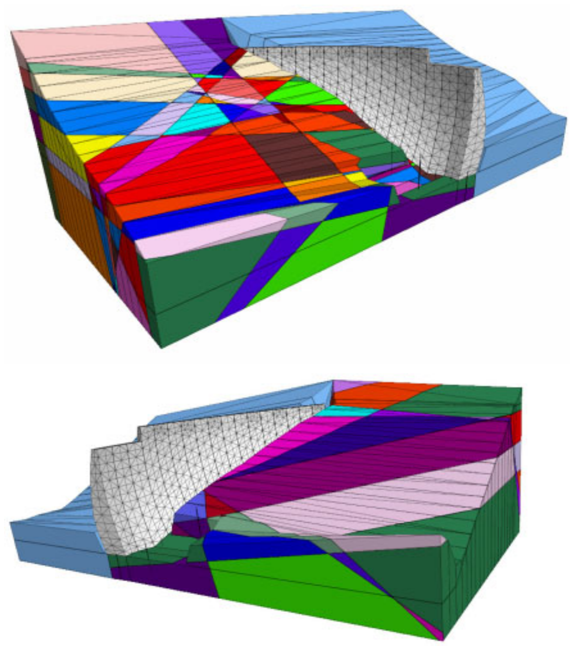

Figure 5. DE block model of Baixo Sabor arch dam. Views of right and left bank halves of model.

In the case of Baixo Sabor dam, 3 main sets were identified in the granitic rock mass, one sub-horizontal and two sub-vertical. The model in Figure 1 includes the main faults found at the site, mostly sub-vertical, whose traces are shown in Figure 4. Only a few joints were selected to represent each set. Figure 5 shows, separately, the right ad left banks of the same model, where the slight upstream dip of the sub-horizontal set is visible.

It should be pointed out that in most figures in this paper, the different rock blocks in the assembly are represented by different colors. These polyhedral blocks are formed by joining convex sub-blocks, with the construction lines also visible. The mesh of tetrahedral elements inside each deformable block is not depicted, for clarity.

It may be seen in these figures that the blocky structure only exists in part of the model. Below and to the sides, beyond the reach of the possible failure modes, large deformable blocks were used. Given the relatively small overall dimensions of the model, the boundary conditions applied at the vertical upstream and downstream boundaries have to be carefully chosen. It is important to use a stress boundary, applying the in situ stress state in the rock mass, instead of the common displacement boundary conditions, to avoid an artificial constraint of upstream-downstream movements. It is enough to fix displacements at the base and sides of the model.

In addition, the discontinuities were also not extended upstream. Besides the computational savings, this is a conservative simplification which prevents the upstream rock to constrain block movements, as often tensile stresses develop at the dam upstream heel. In fact, in this model, a vertical joint was 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 upstream joint is assigned rock joint properties, thus no tensile strength, and water pressures given by the full reservoir conditions are applied. Therefore, part of this joint may separate if the normal stresses become tensile, effectively decoupling the top layer of the upstream rock mass.

3.3.2 *Use of multiple models*

The need to avoid excessive complexity of numerical representations has been often stressed (e.g. Starfield & Cundall 1988). The computational advantages of limiting the number of blocks are today less critical, except perhaps for time domain dynamic analyses. However, the time and labor savings in model generation, model verification and, particularly, in the interpretation of results may be significant.

For dam foundation studies, it is frequently a good option to build several models of the same rock foundation to check different failure modes than to try to include every aspect of behavior into a single complicated representation. For example, it may be possible, and actually more instructive, to study failure modes on each abutment separately. This was done for Alto Ceira dam, a 41m high arch dam under construction by EDP, where the jointing was complex and with different orientation in each valley side (Lemos & Antunes 2011). In fact, 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 6 shows one of these models at the top. At the bottom, the left bank model, 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.

Fairly detailed representations of jointing are possible whenever necessary, as shown in Figure 7, in the model developed by N.S. Leitão to study the stability of the left abutment of Foz Tua arch dam (Matos et al. 2011). In this figure, the vertical discontinuity that limits upstream the blocky representation is visible.

3.3.3 *Persistence of discontinuities*

The representation of non-persistent joint sets in stability calculations still poses some difficulties. Work on joint generation packages has been mostly directed towards fluid flow problems, where network connectivity is more important than partition into blocks. In failure studies, the most conservative option is to disregard the shear strength of the rock bridges, assuming the joints to be continuous. If an acceptable safety margin can be ensured, then there is no need of more elaborate models. If this is not the case, the non-persistence of the joint planes has to be taken into account. If blocks are assumed elastic, then a joint generator that creates non-persistent joint patterns may be unconservative, as small rock bridges may prevent the development of a failure mode. A simpler alternative is to create through-going planar cuts, and simulate rock bridges by assigning cohesive strength to some sections of the joint plane. In this way, the potential the failure of rock bridges and the coalescence of the cracked sections may be contemplated, but joint constitutive models with consistent fracture criteria need to be employed (e.g. Resende et al. 2004). For complex fracture patterns, DE bonded particle models become the most powerful approach, also able to allow fracturing to proceed through the rock blocks, for example, the "synthetic rock mass" concept presented by Pierce et al. (2007).

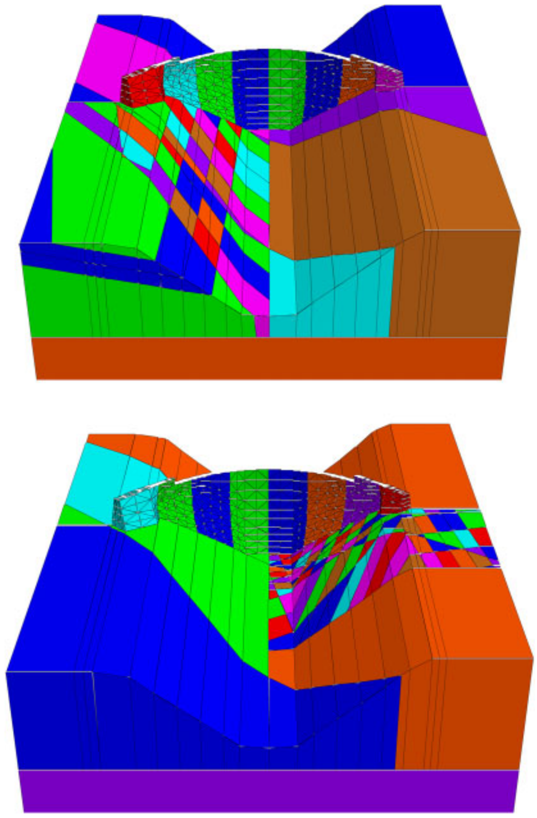

Figure 6. DE block models of Alto Ceira dam, for analysis of right and left bank failure modes.

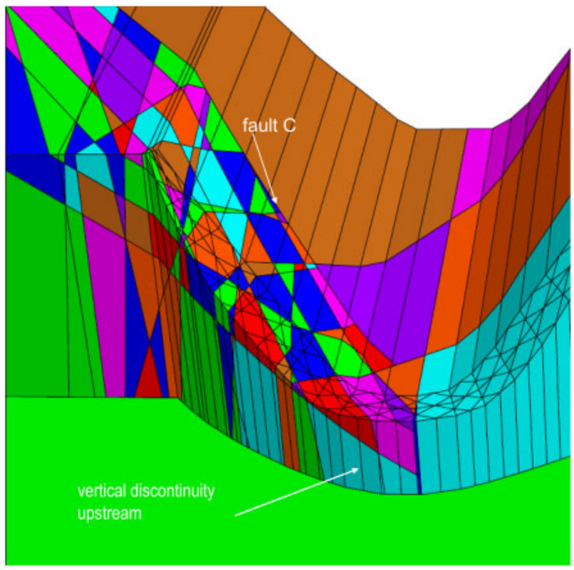

Figure 7. Detail of block model for the analysis of failure of the left abutment of Foz Tua dam (dam and upstream rock hidden).

3.4 *Block and joint deformability*

Modeling the foundation with deformable blocks allows stresses to be evaluated in the rock blocks, but, more importantly, provides a better approximation of the distribution of loads applied by the dam in cases of asymmetrical or heterogeneous rock mass moduli. The patterns of load redistribution after incipient slip are also better judged.

In rigid block models, the overall rock mass deformability is governed only by the joint stiffness. In a deformable block model, both block moduli and joint stiffness are specified. If the real joint spacing were used, then the actual stiffness parameters would be appropriate. However, in a large model of a dam foundation only a few joints may be included, as already discussed. Therefore, the effect of joint stiffness on the overall deformation is small, unless, for example, a thick gouge fault is present. The block moduli need to be selected to provide the correct rock mass deformability for each region. In those cases where the blocky structure only exists in part of the model (e.g. Fig. 6), the effect of the selected joint stiffnesses has to be estimated, so that the moduli assigned to the blocks compensate any imbalance due to the numerical options. As is to be expected, in these stability analyses joint strength parameters are the most decisive properties.

3.5 *Joint water pressures*

The water pressures in the discontinuities are a critical factor in stability analysis. In the assessment of existing dams, monitored piezometric data is very helpful in the calibration of numerical models. For new dams, however, simplified water pressure distributions are usually assumed, according to standard design practices. It is possible to carry out fluid flow analysis with DE models, and several studies have been presented for gravity dams (e.g. Lemos 1999, Barla et al. 2004, Gimenes & Fernández 2006). The code 3DEC also allows an analysis of fluid flow in the rock joints to be performed (Damjanac & Fairhurst 2000), but there is often not enough information to undertake such studies at design stage. In addition, a fracture flow analysis requires a network with many more joint planes than those that are necessary for a failure analysis. Moreover, the grout curtain and the drainage system complicate the flow patterns in the vicinity of the dam. A more practical alternative is to perform an equivalent continuum flow analysis to calculate the field of water pressures throughout the rock mass. 3DEC may also be employed in such equivalent continuum flow analysis, as in a model of Alqueva dam (Fig. 8), where the rock permeabilities were calibrated by monitored pressure and drainage data for various reservoir levels (Farinha et al. 2011). The water pressures obtained in such analyses may then be applied in the joints of the block model used in the mechanical failure study.

In the model of Baixo Sabor dam, simplified water pressure distributions were considered. Along the concrete-rock interface, uplift pressures were prescribed according to the usual design criterion, a bilinear 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.

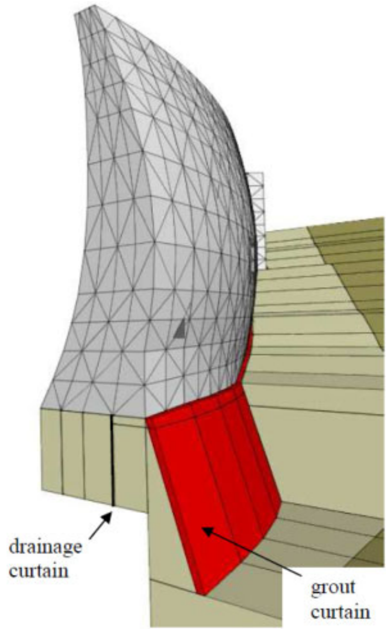

Figure 8. Detail of 3DEC model of Alqueva dam for analysis of water flow (Farinha et al. 2011).

3.6 *Modeling sequence*

3.6.1 *DE model setup*

The definition of the model geometry and the introduction of the rock discontinuities, as discussed, create the block system. A deformable block formulation implies the generation of an internal element mesh in each rock block. For polyhedral block shapes the simpler option is to use tetrahedra, as 3DEC does, which can easily be automatically generated. DE codes typically have routines to detect and update contacts during a large displacement analysis, and these are also invoked to identify the initial contact between the blocks, without user intervention. Contact formulations vary among the different codes, but the most common option is to create contact points at every vertex-to-face or edge-edge interaction (e.g. Lemos 2008). When a block has a fine tetrahedral mesh, grid-points exist within the original rigid block faces, which are also treated as new vertices.

Material properties must then be assigned to concrete and rock blocks and joint properties prescribed for all discontinuities. For systems with complex joint patterns, it is necessary to verify very carefully if the correct constitutive assumptions and properties were assigned to each joint set, as this is the critical factor in the study.

3.6.2 *Modeling steps*

The analysis procedure comprises a sequence of modeling steps, which should as much as possible follow the physical path. The first step corresponds to the in situ condition, before dam construction. In situ stress measurements may provide an estimate of horizontal stresses, but any prescribed initial stress field must then be brought to equilibrium under gravity, ending in a state compatible with the valley shape.

The second modeling step is the simulation of dam construction, in which gravity is applied to the cantilever blocks. The most realistic procedure is typically to assume independent cantilevers, and, at the end, impose the closure of the contraction joints, zeroing joint displacements. The next step is the reservoir filling, with the application of the hydrostatic pressure to the dam upstream face. The water pressures in the discontinuities are also introduced, as discussed in the previous section. Then, the safety assessment procedure follows.

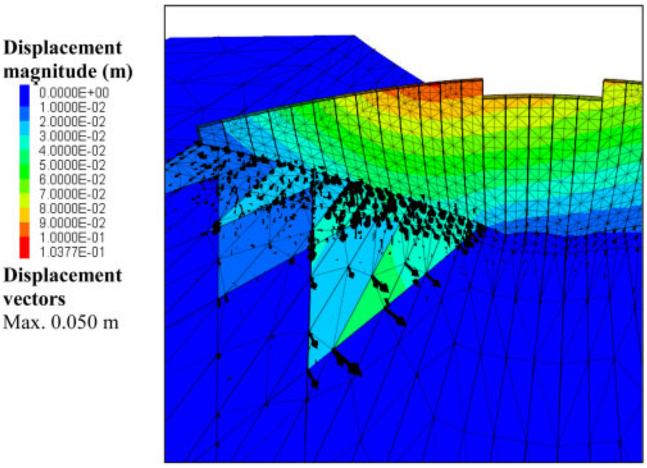

Figure 9. Baixo Sabor dam model. Displacement field for a strength reduction factor of 2.

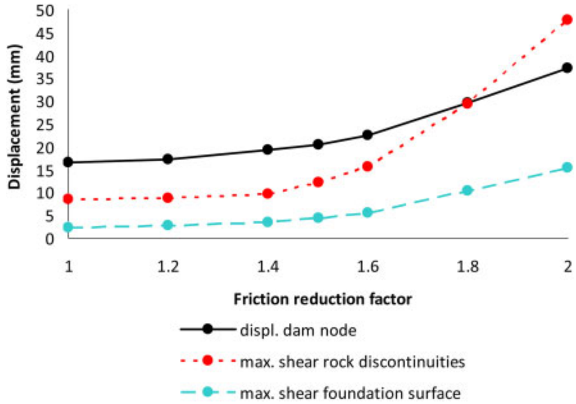

Figure 10. Baixo Sabor model. Evolution of displacement indicators with rock joint friction reduction factor.

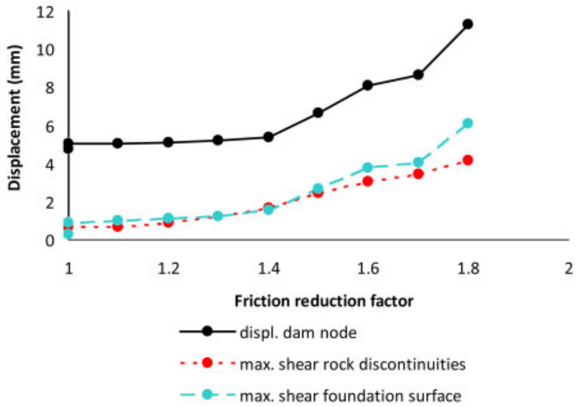

Figure 11. Alto Ceira model. Evolution of displacement indicators with rock joint friction reduction factor.

3.6.3 *Safety evaluation*

The methodology adopted for safety evaluation must satisfy regulatory requirements, a comprehensive survey and discussion being given in ICOLD European Club (2004). In the studies presented in this paper, safety factors were calculated by means of a strength reduction procedure. The shear strength of the discontinuities, where nonlinear behavior is concentrated, is divided by progressively larger factors until collapse takes place or displacement magnitudes reach 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. The treatment of concrete-rock interface varies with national regulations. For example, the Portuguese code requires that a condition of no cohesion or tensile strength be checked for failure scenarios.

In the case of Baixo Sabor dam (Figs. 1–5), shear tests of rock joints led to friction angles of 37° to 39° for the 3 main sets. Faults were assigned a friction angle of 35°, and the rock-concrete interface a value of 45°. The orientation of the discontinuities provided a comfortable safety margin. The development of a failure wedge under the right abutment, shown in Figure 9, was only obtained for a friction reduction factor of 2.

Figure 10 contrasts the evolution of 3 displacement indicators during the friction reduction process. The displacement of a grid-point in a rock block under the right abutment (solid line) starts increasing at a reduction factor of about 1.6, a trend displayed more clearly by the curve corresponding to the maximum shear displacement of any rock discontinuity in the model (short dash line). The lower curve (dashed line) represents the maximum shear displacement at the concrete-rock interface, assumed cohesionless in this run, also follows the trend, but less expressively. These results show that slip on the rock wedge surfaces may not be immediately noticeable on the structural displacements.

The Alto Ceira right bank model (Fig. 6), also displays a significant safety margin, but a different type of response is illustrated by the corresponding curves. The displacement indicators in Figure 11 show that the joint slip starts affecting dam displacements when the reduction factor exceeds 1.4, and a clear acceleration can be seen above 1.7. In conclusion, for a correct interpretation of the numerical model output, it is important to look at multiple indicators, namely the block movements and joint slip at several points, to detect the onset and progression of the failure modes. Often, it is not advisable to expect a complete collapse of the block model, as it is possible that equilibrium is reached with displacements unacceptable for the integrity of the concrete structure, or for the continued effectiveness of the grout and drainage curtains.

4 CONCRETE GRAVITY DAM ANALYSIS

4.1 *Sliding failure modes*

For concrete gravity dams, the most common failure scenarios to be evaluated involve sliding on the rock-concrete interface or on shallow sub-horizontal rock joints. The common practice is to analyze the stability of individual dam blocks, assumed to move in the upstream-downstream direction, but 3D analysis may sometimes be advisable, as discussed below. At the valley slopes, 3D collapse modes, as considered for arch dams, may be possible. DE models allow the evaluation of all these failure scenarios, whether for static or seismic loads. They may also be employed to check the safety against sliding on the dam horizontal lift joints, also a concern for concrete gravity dams. In all of these calculations, the water pressure distribution has a crucial influence on the outcome.

4.2 *2D vs. 3D analysis*

The analysis of sliding failure modes of gravity dams is usually performed in 2D, assuming the dam blocks to behave independently, thus neglecting the possible contribution of shear keys. This is sometimes even done for dams with a slight curvature in plant, in which the extra safety provided by the arch is not taken into account. The 3D effect, however, is clearly present in the case of narrow valleys, even for straight axis dams. In studies of safety re-evaluation of older dams, if the shear keys are capable of providing a monolithic behavior, this extra contribution may be valuable (e.g. ICOLD European Club 2004).

When sliding on rock joints is also analyzed, the 3D nature of rock mass structure always exerts some influence. Even if the movement of a single dam block can only take place in the upstream-downstream direction, the water flow pattern, for example, is less well captured by the plane analysis assumptions.

Analytical methods, based on limit equilibrium techniques, have also been proposed for 3D gravity dam analysis (e.g. Lombardi 2007; Sun et al. 2010). These simplified techniques are more often applied to the study of sliding on the dam-foundation interface. DE numerical models not only provide a full mechanical analysis tool, but are also more versatile if the rock mass joints need to be included.

4.3 *Seismic analysis*

Seismic action is always a major concern in sliding failure scenarios for gravity dams. The shortage of historical data on the response to large seismic events makes experimental tools of great value in the validation of numerical models. Figure 12 show a model of a concrete dam monolith tested on LNEC's shaking table (Gomes 2006). A blocky foundation was created by 2 joint sets, allowing several mechanisms to be simulated, by means of changing the friction on the various joints, and locking or not the joint representing the dam-rock interface. The test results were then analyzed by a 3DEC model with deformable blocks.

Often, the seismic stability of a gravity dam is analyzed by very simple 2-block models, in which the dam is represented by a rigid block and the rock mass by another with a prescribed acceleration record. It should be emphasized that this is a very crude model, acceptable only as a first check on the likelihood of slip under the earthquake action. If this model indicates that slip is possible, then a detailed numerical model needs to be built, with proper boundary conditions for seismic analysis (Lemos 1999, 2008). The dam rocking modes need to be correctly represented, as well as the foundation deformability and joint water pressure distribution, which greatly influence the sliding behavior.

The evolution of the joint water pressures during a dynamic event is another important topic, and experimental results such as those obtained by Javanmardi et al. (2005) are very valuable. Dynamic numerical simulations (e.g. Lemos 1999) have indicated that keeping the steady-state pressure field in the joints, thus not taking into account the dynamic water pressure variations, may be a conservative assumption, leading to larger slip predictions. This issue needs experimental confirmation, but it would facilitate seismic analysis, especially since the effective water stiffness depends on joint apertures, always difficult to characterize.

For large design earthquakes, the simplifying assumptions imposed by some regulations, such as a cohesionless dam-rock interface, lead to significant permanent displacements. A consistent estimate of these displacements is required, and further investigation is needed on criteria for the definition of acceptable levels of displacement, and their potential effects on structural safety (Alliard & Léger 2008).

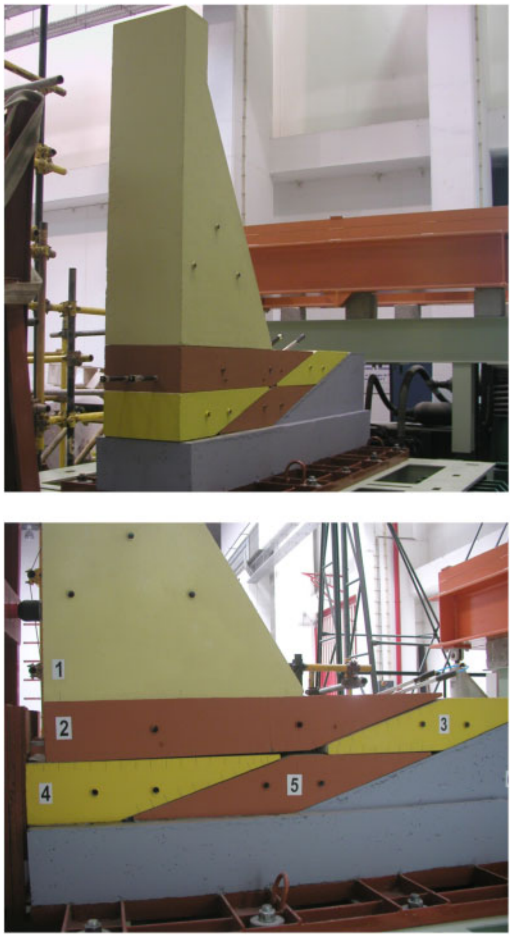

Figure 12. Gravity dam model on blocky foundation for shaking table test of (top); permanent joint displacements after test (bottom) (Gomes 2006).

4.4 *Modeling and numerical issues*

The application of DE deformable block models to the failure analysis of gravity dam foundations follows, in essence, the methodology outlined for arch dams. A 2D model is much easier to create, and faster to run, particularly in dynamic analysis. Therefore it is almost always a good starting point, even if a 3D model becomes necessary for more rigorous predictions.

For sliding on the dam-rock interface, planar models are usually sufficient, and the key role is played by uplift pressures. When assessing existing dams, piezometric readings provide essential information. In 2D models, the analysis of water flow in the fractures poses no computational difficulty, unlike the case of arch dams previously discussed. Various studies with DE codes have been published (e.g. Lemos 1999, Barla et al. 2004, Gimenes & Fernández 2006). Of course, the questions about the suitability of the fracture networks remain, as well as those on the reliability of in situ joint conductivity data. It is known from various numerical studies that flow simulations are much more dependent on the chosen jointing patterns, and exhibit more scattering of results, than mechanical stability calculations.

The explicit solutions algorithms employed by most DE codes have proved fairly robust for strongly nonlinear conditions, as experienced during failure analysis of blocky media.

For dynamic analysis, these algorithms call for small time steps, making large 3D systems still a computational challenge. An experienced user, however, aware of the specific numerical stability constraints implemented in the code, is often capable of building the model in a way that achieves substantially reduced run times.

The comments made above regarding the need for consistent fracture criteria in the analysis of rock bridges also apply to the study of the concrete-rock contact. A good deal of research has been published on this subject, either using fracture mechanics approaches or slip-weakening joint models, namely at ICOLD's numerical benchmark workshops.

5 CONCLUSIONS

The numerical models presented provide a powerful tool to address the safety assessment of structural foundations on rock. The current DE models based on deformable block formulations have all the features required to analyze arch or gravity concrete dam foundations, as discussed in the previous sections. At a first level of analysis, they may substitute traditional techniques of limit equilibrium, overcoming specific restrictive assumptions. For this purpose, the natural tendency to overelaborate a numerical representation should be resisted. For more complex applications, in order to take advantage of all the capabilities of these numerical models, it is necessary to have enough reliable experimental data.

An effective use of these models in safety assessment requires continued investigation on a range of issues, extending from the strictly numerical aspects, such as reducing the computational cost of large systems, to the more essential matters of representing rock and joint behavior in ultimate scenarios. Among those issues, a few may remain central in dam foundation failure studies: practical joint generation techniques specifically intended for stability analysis; a consistent and dependable treatment of rock bridge or dam-rock interface fracture; estimation of joint water pressure distributions providing a reasonable envelope for specific in situ conditions.

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