XI International Conference on Computational Plasticity. Fundamentals and Applications
COMPLAS XI
E. Oñate, D.R.J. Owen, D. Peric and B. Suárez (Eds)

NUMERICAL ANALYSIS AND SAFETY EVALUATION OF A LARGE ARCH DAM FOUNDED ON FRACTURED ROCK, USING ZERO-THICKNESS INTERFACE ELEMENTS AND A c-ϕ REDUCTION METHOD

I. ALIGUER*, **I. CAROL*** **AND E.E. ALONSO***

* Geotechnical Engineering and Geo-Sciences Department (DETCG)
ETSECCPB (School of Civil Engineering)
UPC (Universitat Politécnica de Catalunya), 08034 Barcelona
e-mail: ignasi.aliguer@upc.edu, ignacio.carol@upc.edu, eduardo.alonso@upc.edu

Key words: FEM, Rock Mechanics, Strength Reduction, Slope Stability

Abstract. A 140m high arch dam in the Pyrenees, built in the 50s, is founded on fractured limestone rock. Since the beginning of the design process, two main families of discontinuities were identified. The dam was built very close to the end of the narrow part of the valley, which raised stability concerns early on. In the late 80s - early 90s, a numerical study of the dam was developed at the Dept of Geotechnical Engineering and Geo-Sciences UPC (School of Civil Engineering) UPC, using a progressively more realistic series of models and approaches, culminating with a 3D discretization of the dam plus rock mass, in which discontinuities were explicitly represented using zero-thickness interface elements with frictional constitutive laws in terms of stress tractions and the corresponding normal and shear relative displacements. In the present study, that dam and its foundation are revisited and reanalyzed with current, more advanced numerical tools and a third family of rock joints which has been identified more recently. The same mesh is used as a departure point, although a much more detailed description is now possible. The analysis is also approached in a different way, now using the traditional $c-\phi$ reduction method developed and implemented specifically for non-linear zero-thickness interfaces.

1 INTRODUCTION

Canelles dam is a 151m high arch dam loacted in the Pyrennes (Catalunya, Spain), which was completed in April 1958. Since this date, different kinds of analyses have been carried out. Monitoring systems, model tests and numerical analyses have been combined to provide engineering evaluations of the dam safety [3, 4]. The most important one, concerning the stability of the dam and both abutments, was performed during the 90s decade. The results and analysis procedures are reported in several references. [1, 2, 9, 13]

The non-linear calculation was carried out for real values of the gravity and water pressure loads, and safety was evaluated a posteriori by post-processing the stresses obtained along all interface families. A number of potential failure mechanisms were selected, and for each of them resisting forces and acting forces were evaluated, and safety coefficients were obtained by assuming a proportional increase of stress tractions with increasing water pressure or a decreasing resistance given by lower friction angle. That first study led to the conclusion that the dam was basically safe and the worst scenario corresponded to a safety coefficient between 2 and 3.

The dam is founded on cretaceous massive limestone that is fractured by two sets of discontinuities (Fig. 1). A main set of vertical joints is oriented parallel to the valley. The other family is a set of N-S planes which dip an average of 55° towards the West (nearly downstream). A Laser-Scanner field campaign in 2009 led to the identification of a third set of discontinuities. In addition, bedding planes dip 45° upstream. Due to the spatial arrengement of the three rock discontinuity families, several rock blocks have fallen down to the canyon, which keeps the facilities and people in danger. That has motivated to consider a new retaining wall in the left abutment. As a consequence, an analysis, this time focused only on the left abutment, has been recently started to evaluate the safety improvement that could be achieved with the construction of the wall.

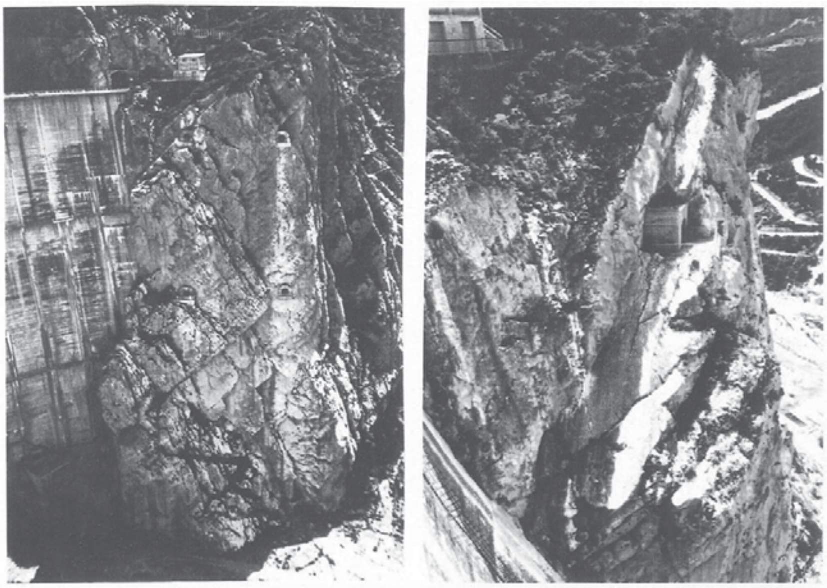

Figure 1: General view of the left abutment and anchorage tunnels

According to the 3D geometry of the canyon and the arch dam, the Finite Element Method is required to reach the analysis purpose. Considering the new set of joints and other constructive details such as the new wall and four existing anchorage tunnels, the former model has been rebuilt. Moreover, block equilibrium analysis has been performed as a first simplified evaluation of the safety factor, which involves the retaining wall.

Safety analysis in engineering practice often requires simplifications such as the concept of safety factor, which tries to provide a single scalar simplified mesure of the distance between the failure state and service conditions.

2 MODELING THE LEFT ABUTMENT

2.1 Geometric model

As described in [1], a safety 3D FEM analysis has a number of requirements, including sufficient number of elements across the dam thickness in order to capture bending, and a sufficient number of joints of each family in the rock mass in order to include the most relevant failure mechanisms.

For the current analysis, the previous discretization has been verified with digital satellite topodata, and has been used as a basis fot the new geometric model, including: dam geometry, rock mass surface topography, right abutment retaining wall, grouted curtain and bedding and vertical joint sets. Additionally, left abutment retaining wall, reinforced concrete anchorage tunnel and 4 new family planes have been introduced only in the left abutment. The new family of discontinuities discovered, has been introduced in the geometric model through a total of four planes strategically located so that they cover the most significant mechanisms without generating an excessive number o geometric intersections with the previous existing planes and surfaces. The four selected planes are:

1. Plane defining a mechanism that does not involve the wall projected.
2. Plane intersecting the dam's top.
3. Plane crossing the new wall's base.
4. Plane defining a mechanism that is not supported by the anchorage tunnels.

Finally, in terms of rock discontionuities the model includes: 12 vertical joints (jV 1-12), 5 bedding planes (jS 1-5) and 4 new family planes -N-S orientation- (jN 1-4). (Fig. 2)

2.2 Material Parameters

The shear strength parameters were obtained from the existing information form large scale in situ shear tests on vertical joints ($c = 0.124MPa$ and $\phi' = 18.7°$) and bedding planes ($c = 0.135MPa$ and $\phi' = 35.2°$) [1]. For the new family planes, an anular shear test of the infill clay was performed yielding a residual strength value of $\phi'_{res} = 27°$.

Normal and shear stiffness moduli of all joints have been taken high as usual in numerical analysis with zero-thickness interfaces, in order to ensure the continuity of both sides of the joint if the shear stress does not reach the shear strength values ($K_n = K_t = 10^5 MPa$).

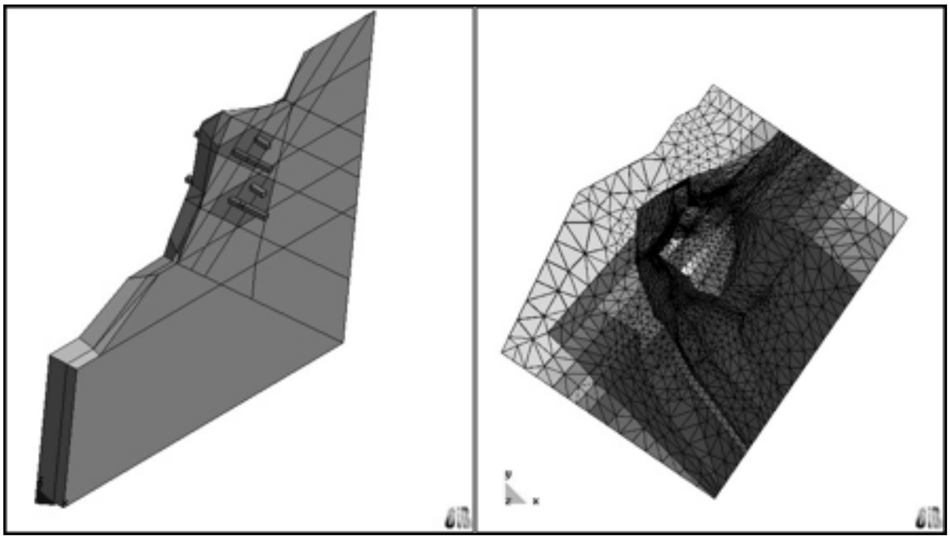

Figure 2: Vertical joint section (left). FEM mesh for the left abutment (right)

3 FINITE ELEMENT ANALYSIS

3.1 FEM Code

FEM computations have been performed using code DRAC [5, 14]. This is an in-house developed code which was first applied to rock mechanics problems considering zero-thickness interface elements, and later also used for fracture mechanics and a variety of other material and structural analysis applications. Its main flow diagram consists of 4 nested loops which correspond to:

Stage loop In each stage new geometry can be added (construction) or removed (excavation).

Step loop Load systems can be applied to each geometry.

Increment loop Each load can be applied in increments.

Iteration loop In non-linear analysis, this loop controles the number of iterations to convergence.

3.2 Interface constitutive model

The constitutive model which was implemented in the code and it is the most widely used for geotechnical analysis. It is a general elastoplasticity law which was formulated in terms of normal and shear stress and normal and tangential relative displacements [7]. However, a simplified version exists that permits an explicit integration [8]. The main simplifying assumptions are:

1. Perfect plasticity.
2. No dilatancy.
3. Linear elastic relationship between the normal stress and the normal relative displacement in compression (zero normal stress in tension).

The yield surface in the $\sigma - \tau$ plane, where $\tau = \sqrt{\tau_1^2 + \tau_2^2}$, is defined by (Fig. 3):

$$F = \tau^2 + \tan^2\phi(\sigma^2 + 2a\sigma) \equiv 0 \quad (1)$$

Due to the expression of the yield surface and the elastic relationship between the normal stress and the normal relative displacement, once the normal stress is known, the ratio between τ_1 and τ_2 is the only unknown in the integration of the constitutive law. The θ angle, which represents this ratio can be obtained by solving the following differential equation:

$$\frac{d\theta}{\sin(\beta - \theta)} = \frac{K_t}{K_n}\frac{\Delta v}{\Delta u}\frac{d\sigma}{\tan\phi\sqrt{\sigma^2 + 2a\sigma}} \quad (2)$$

Integrating between two instants leads to:

$$\tan\left(\frac{\beta - \theta}{2}\right) = \tan\left(\frac{\beta - \theta_0}{2}\right)\left(\frac{\tau + \sqrt{\tau^2 + a^2\tan^2\phi}}{\tau_0 + \sqrt{\tau_0^2 + a^2\tan^2\phi}}\right)^{-\frac{K_t}{K_n}\frac{\Delta v}{\Delta u \tan\phi}} \quad (3)$$

where β relates the imposed tangential relative displacements increments $\Delta v_1, \Delta v_2$.

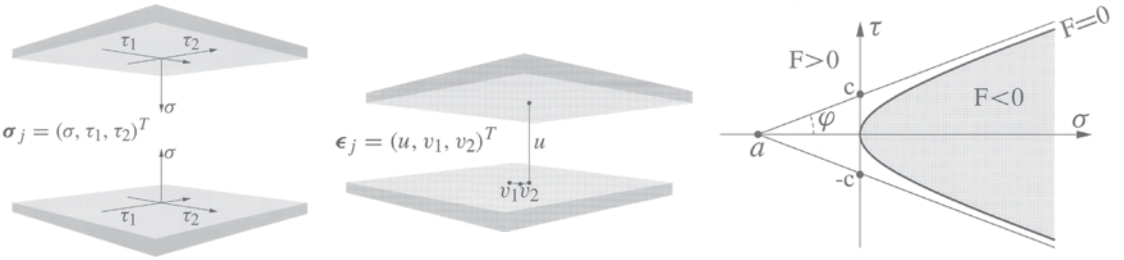

Figure 3: Constitutive model stress and displcement variables definition (left, center). Yield surface (right)

3.3 $c - \phi$ reduction

The first reference mentioning the idea of reducing the strength parameters of the material to evaluate the Safety Factor seems to be that of Zienkiewicz et al., 1975 [15], for a slope stability problem in soil. Later, various authors have used this method for other soil mechanics problems [6,10,12] Generally speaking, the Safety Factor (SF) is defined as the scalar factor by which one has to reduce strength parametres in order to reach failure. Hence, at failure, the following relationships can be stablished:

$$c'_{mob} = \frac{c'}{SF} \qquad \phi'_{mob} = \arctan\left(\frac{\tan\phi'}{SF}\right) \tag{4}$$

In the numerical analysis of geotechnical problems using the FEM there are a few ways to define failure, but the most common (also used in this case) is the lack of convergence of the iterative calculation. In this study, the $c-\phi$ reduction method has been implemented as a modification of the constitutive model of section 3.2 with evolving c and ϕ. In this new implementation c and ϕ are being progressively reduced (softening) in connection to some fictitious time α:

$$\tan\phi \equiv \Phi = \Phi_0 - \frac{\Delta\Phi}{\Delta\alpha}(\alpha - \alpha_0) \tag{5}$$

Therefore, the right hand side of the ecuation (2) can be reformulated using this variable:

$$\frac{K_t \Delta v}{\Delta\alpha} \frac{d\alpha}{(-f\alpha + g)\sqrt{A\alpha^2 + B\alpha + C}} \tag{6}$$

where:

$$f = \frac{\Delta\Phi}{\Delta\alpha}$$
$$g = \Phi_0 + \frac{\Delta\Phi}{\Delta\alpha}\alpha_0$$
$$A = \left(\frac{\Delta\sigma}{\Delta\alpha}\right)^2$$
$$B = 2\left(\sigma_0\frac{\Delta\sigma}{\Delta\alpha} + a\frac{\Delta\sigma}{\Delta\alpha} - \alpha_0\left(\frac{\Delta\sigma}{\Delta\alpha}\right)^2\right)$$
$$C = \sigma_0^2 + 2a\sigma_0 - 2\sigma_0\alpha_0\frac{\Delta\sigma}{\Delta\alpha} - 2a\alpha_0\frac{\Delta\sigma}{\Delta\alpha} + \alpha_0^2\left(\frac{\Delta\sigma}{\Delta\alpha}\right)^2$$

In the same way as the previous section an analytical integration can be done to obtain the angle θ.

3.4 Verification examples

As a first example, a classic rock slope stability problem has been considered [11]. The geometry is represented in Fig.4. For this case, using the Limit Equilibrium Method the Safety Factor can be easily determined using a Mohr-Coulomb type failure criteria:

$$SF = \frac{T_{res}}{T_{mob}} = \frac{2c}{\gamma H \sin\beta\cos\beta} + \frac{\tan\phi}{\tan\beta} \tag{7}$$

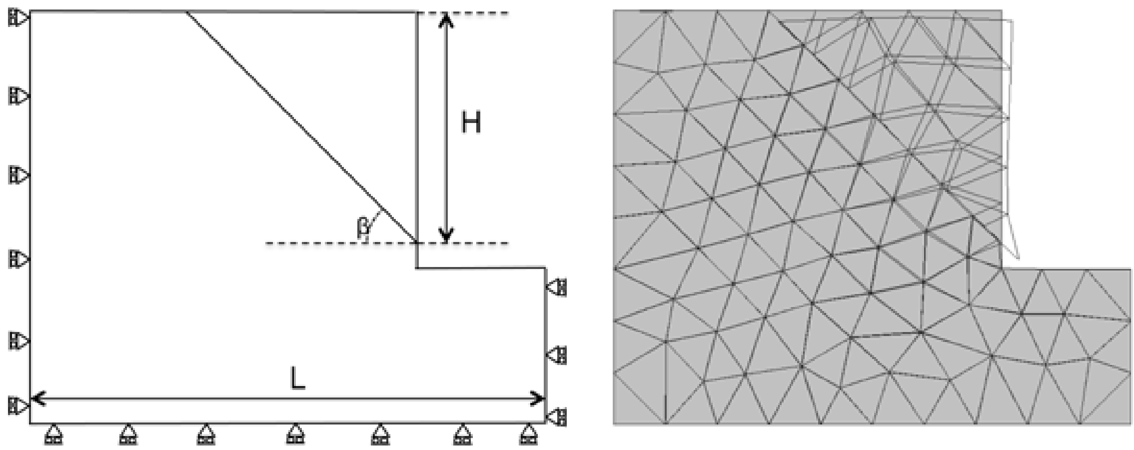

Figure 4: Geometric definition of the rock slope, FEM mesh original and deformed at failure.

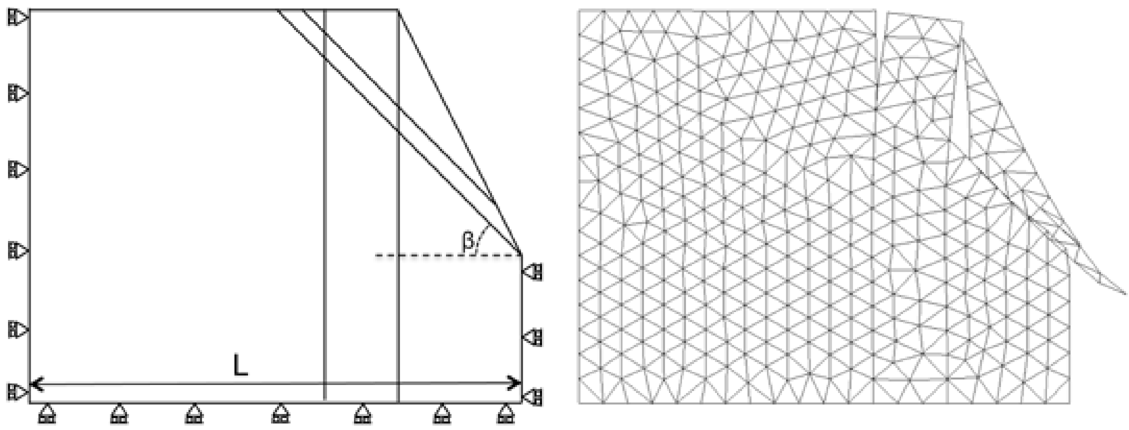

Figure 5: Geometric definition of the rock slope, FEM mesh deformed at failure.

The numerical example is with the following dimensions and parameters: L=100m, H=45m, $\beta = 45°$, $\tan\phi_{res} = 1.4$, c=10kPa (for both Mohr-Coulomb Limit Equilibrium and hyperbolic model). Introducing all these values into (7) a Safety Factor of 1.43864 results.

In the FEM computation a reference increment of $\Delta \tan\phi/\Delta t = -0.05$ was imposed, and the failure occured for a value of $\tan\phi_{mob} = 1.065$. Hence, a Safety Factor of 1.315 can be obtained using the $c - \phi$ reduction method.

In this case, as the rock matrix was considered linear elastic, the failure could only happen by sliding along the one single rock joint considered. Nevertheless, when more discontinuities are considered the failure mechanism turns out not so obvious.

The second example is also a 2D rock slope stability, but for the case of four joints intersecting each other. As a result, failure mechanisms that involve more than one joint can arise and no simplified formula such as (7) can be used. As can be observed in Fig.5, two dipping and two vertical joints are considered. The deformed mesh at the end of the calculation reveals that the failure mechanism involves the lower inclined plane and the first vertical.

For that second example the same geometry was taken: L=100m and $\beta = 45°$ and the initial strength parameters were: c=10 kPa and $\tan\phi_{res} = 1.5$. Also a reference variation of $\Delta\tan\phi/\Delta t = -0.05$ was imposed, and the failure occured when the $\tan\phi$ had a value of 1.118.

3.5 Application to the left abutment of Canelles Dam. Preliminary 2D results

The last example presented in this paper consists of the application of the c-phi reduction method to the stability of the left abutment of Canelles Dam. As a first step in this direction, a 2D vertical cross-section of the rock mass has been extracted from the new 3D geometric model described in section 2. The aim of this preliminary analysis is two-fold: (1) to verify the method performance in a real case and (2) to evaluate the increase of the Safety Factor provided by the construction of the new wall. The vertical cross-section considered is oriented along the maximum dipping direction of the new joint family and intersects the new wall approximately at its gravity center (Fig.6).

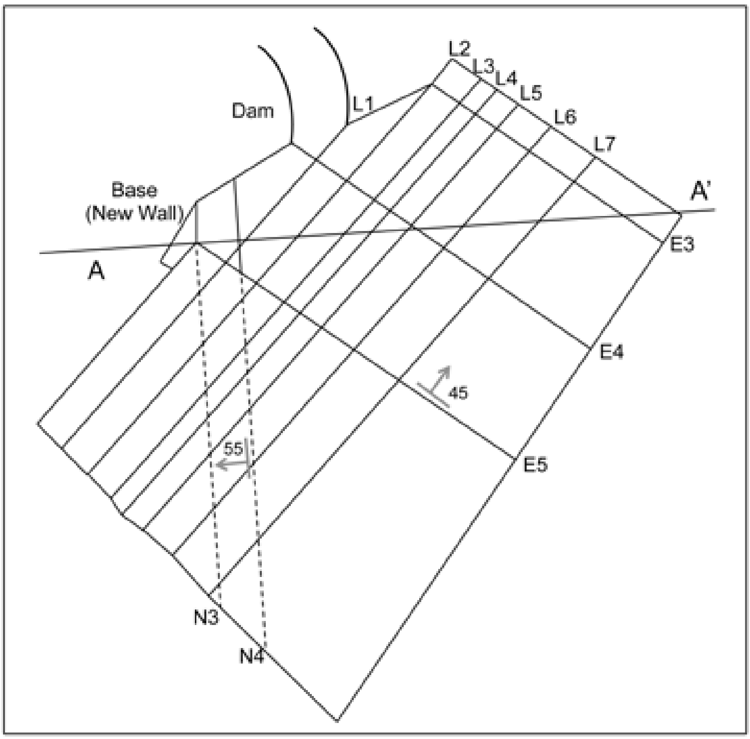

Figure 6: Horizontal cross-section at 375m. Bedding planes (E-planes), vertical principal joints family (L-planes) and new joints family (N-planes) are represented.

As it can be observed in Fig.7, three joints have been included in the discretisation: one representing the new joint family dipping 55°, and two more representing the rock-concrete contact joints (at the base and vertical side of the wall).

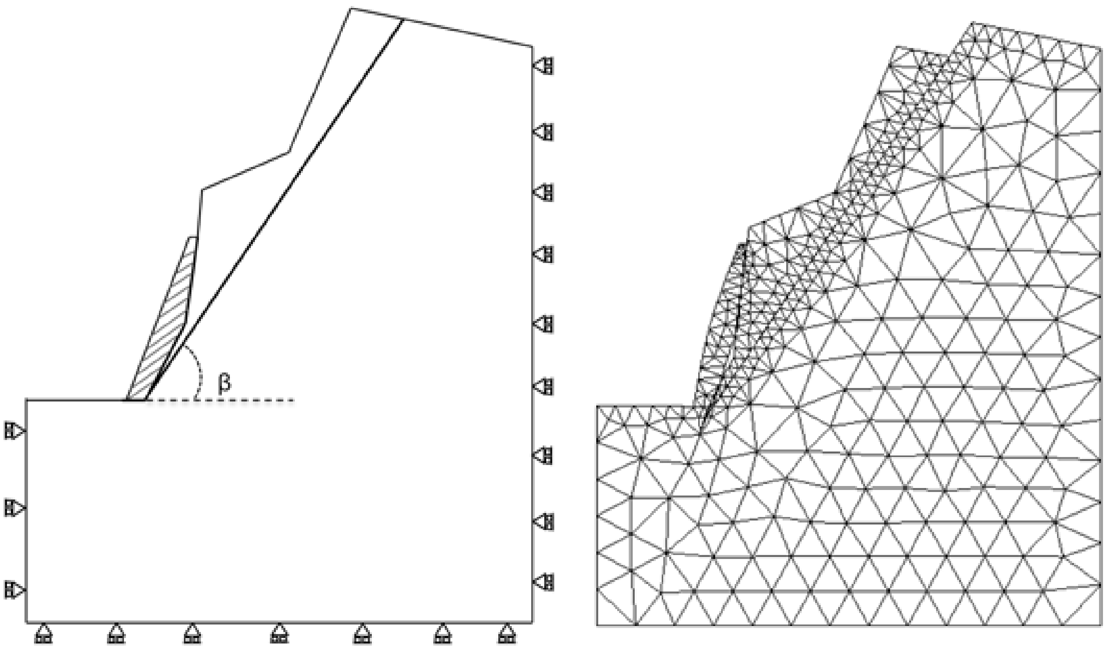

Figure 7: Geometric definition of the rock slope, FEM mesh deformed at failure.

For this example, the initial strength parameters used are: $c = 475$ kPa and $\tan\phi_{res} = 0.7$ (rock-rock), and $c = 10$ kPa and $\tan\phi_{res} = 1.0$ for the rock-concrete contacts. During the $c - \phi$ reduction process, only the rock-rock interface parameters were reduced (that is, those of the new joint family), while those of rock-concrete contact were assumed to remain constant. The failure occured when the $\tan\phi$ had a value of 0.4845, which corresponds to $\phi = 25.85°$.

4 ON-GOING WORK AND CONCLUDING REMARKS

The results presented show that the procedure implemented is capable of leading to reasonable results of failure analysis in rock masses, via $c - \phi$ reduction method in 2D vertical cross-sections.

Current on-going effort is devoted is devoted to (1) analysing new 2D cross-sections incorporating more joints of various families, and (2) carrying out the full 3D calculations of the left abutment of Canelles Dam including the three joint families and anchorage tunnels, with and without the new retaining wall. The latter case might require the improvement in the efficency and robustness of the numerical techniques employed.

5 ACKNOWLEDGMENTS

This research is supported partially by grants BIA2009-10491, from MICINN (Madrid,Spain), 2009SGR-180 from AGAUR-Generalitat de Catalunya (Barcelona) and by ENDESA (Madrid, Spain). The first author also wishes to acknowledge his current FPU doctoral fellowship from MICINN.

REFERENCES

[1] E.E. Alonso, I. Carol, C. Delahaye, A. Gens, and P. Prat. Evaluation of safety factors in discontinuous rock. *Int. J. Rock Mech. Min. Sci. and Geomech.*, 33:513–537, 1996.

[2] E.E. Alonso, A. Gens, I. Carol, P. Prat, and E. Herrero. Three-dimensional failure mechanisms in arch dam abutments. a safety study. In *XVIII Congres des Grands BArrages*, Durban, 1994.

[3] A. Alvarez, E. Herrero, and J.M. Buil. Some considerations concerning the effect on overall safety of interface problems between arch dams and their foundation. In *XIII Congres des Grands BArrages*, pages 1169–1185, N. Delhi, 1979.

[4] A. Alvarez, E. Herrero, and M. Vizcaino. Dams in karstified zones. three different solutions. In *XV Congres des Grands Barrages*, pages 1225–1246, Lausanne, 1985.

[5] I. Carol and P. Prat. Drac, finite element code. Technical report, Dept. of Geotechnical engineering, ETSECCPB-UPC, Barcelona, 2005.

[6] Y.M. Cheng, T. Lansivaara, and W.B. Wei. Two-dimensional slope stability analysis by limit equilibrium and strength reduction methods. *Computers and Geotechnics*, 34:137–150, 2007.

[7] A. Gens, I. Carol, and E.E. Alonso. An interface element formulation for the analysis of soil-reinforcement interaction. *Computers and Geotechnics*, 7:133–151, 1989.

[8] A. Gens, I. Carol, and E.E. Alonso. Rock joints: Finite element method, implementation and applications. In A.P.S. Selvadurai and M.J. Boulon, editors, *Mechanics of Geomaterial Interfaces*, pages 395–420. Elsevier, 1995.

[9] A. Gens, P. Prat, E.E. Alonso, and C. Delahaye. Finite element analysis for the safety evaluation of a dam on a fractured rock foundation. In *Beyond 2000 in Computational Geotechnics-10 Years of Plaxis International*, Balkema, Rotterdam, 1999.

[10] D.V. Griffiths and P.A. Lane. Slope stability analysis by finite elements. *Geotechnique*, 49(3):387–403, 1999.

[11] E. Hoek and J.W. Bray. *Rock Slope Engineering.* Taylor and Francis, 1974.

[12] T. Matsui and K.C. San. Finite element slope stability analysis by shear strength reduction technique. *Soil and Foundation*, 32(1):59–70, 1992.

[13] P. Prat, C. Delahaye, A. Gens, I. Carol, and E.E. Alonso. Safety evaluation of an arch dam founded on fractured rock based on a 3d nonlinear analysis with joint elements. In *International Conference on Dam Fracture and Damage*, chambery, France, 1994.

[14] P.C. Prat, A. Gens, I. Carol, A. Ledesma, and J.A. Gili. Drac: A computer software for the analysis of rock mechanics problems. In *Aplicatioms of Computer Methos in Rock Mechanics*, pages 1361–1369, Xian, China, 1993.

[15] O.C. Zienkiewicz, C. Humpheson, and R.W. Lewis. Associated and non-associated visco plasticity and plasticity in soil mechanics. *Geotechnique*, 25:671–689, 1975.