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# A method of statistical reconstitution of the fracture network for numerical studies of rock foundations

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**ABSTRACT:** For the design of foundations on rock, the part of numerical modeling becomes essential with the increase in complexity of construction works, especially when the structure of the rock mass is involved. The lack of information on rock discontinuities makes it difficult to reach enough precision in modeling. Thus, we have to resort to a stochastic approach of fractures from borehole data, here gathered in a viaduct project. To illustrate the differences between two approaches, we present a numerical study of the deformation of the rock around a foundation with a continuous modeling and a discontinuous modeling.

## 1 INTRODUCTION AND PROBLEMATIC

The purpose of an artificial structural element referred to as a foundation is twofold: (1) to receive and transmit structural and external loads to the underlying rock at a given depth below the ground surface, safely without causing any distress to the superstructure; and (2) to distribute pressure at the base of the footing of the foundation to an intensity allowable on the rock. Concerning the first point, this paper will not address the effect of external loads as wind or seismic shocks. A foundation must be designed so that the resultant load exerted on it does not cause any lateral sliding of the foundation base on rock, ground collapse underneath the structure, tilting or intolerable settlement of the structure. Deep foundations are laid at considerable depth below the lowest part of the superstructure, to rest on, and to transmit structural loads to a firm geological stratum of adequate bearing capacity.

To determine the limit loads for rock foundations, estimates resulting from empirical studies and on the nonlinear criterion of failure from Hoek and Brown (Hoek and Brown, 1980) are often proposed. This method, supplemented by theoretical assumptions, allowed the development of more specific procedures of application for various types of rocks depending on their state of fracturing (Serrano and Olalla, 1996).

To identify the class of qualification of the rock on the level of the foundations, the internal structure of a rock mass is hardly accessible. Tools of structural geology allow identifying the main discontinuity sets with the help of field data. Further information on the nature and fractures of the rock mass is given by boreholes, but these data are very local and

the real fracture size and shape remain unknown. To deal with this difficulty, we use stochastic algorithms to simulate many realizations of a fracture network conditioned by measured information (Fouché et al., 2001). The computing tool used in this study was designed by the French Institute for Petroleum (IFP). The fracture network is constrained by an a priori model where the geological medium is stratified and heterogeneous. From the spatial coordinates of the intersections between fractures, we reconstitute a 3D system of rock blocks. Then, we intersect this system with a cross section giving a 2D network of fracture traces.

Comparing estimates made from a specific standard for viaduct foundations on rock slopes with those made with the help of numerical models for continuous media (3D or 2D) would be a first task which is not be addressed here. Then, the comparison with the results of discontinuous medium modeling is not direct (Alfonsi and Fouché, 2003). In this paper, we present the case of the viaduct of Saulières (France) to compare the rock mass deformation under the foundations predicted by a continuous numerical modeling (finite elements or finite differences) and discontinuous modeling (distinct elements).

## 2 FOUNDATION ON A ROCK SLOPE

### 2.1 *The different scales of fractures*

The description of a rock mass involves multiple scales: figure 1 sums up the complexity of the structure of a rock slope (Rachez, 1997). The anisotropy of both the fracture network and the rock constitu-

tive materials, as well as the filling of joints, is likely to influence the mechanical behavior of the rock mass.

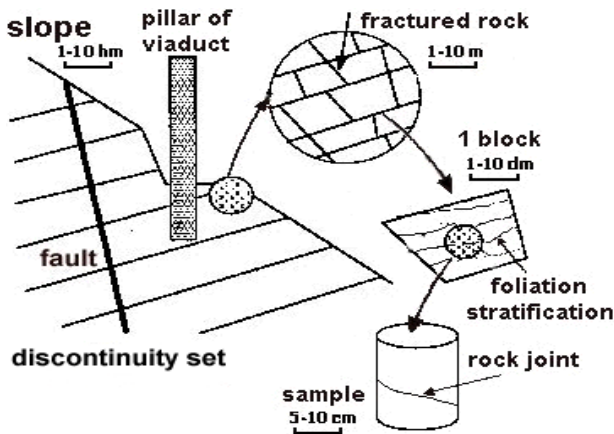


Figure 1. The rock mass at different scales.

In a volume large enough where the multiple order discontinuities do not favor any potential failure surface, we can represent the rock mass as an equivalent continuum (cf. section 3). Properties for materials should be chosen by taking into account the fracture density (Fouché et al., 2003). At a specified scale characterized by a sufficiently well known system of discontinuities (Fouché et al., 2002), another method is to represent explicitly the rock mass as a system of homogeneous blocks (section 4).

## 2.2 The viaduct of Saulières, a rock-founded bridge

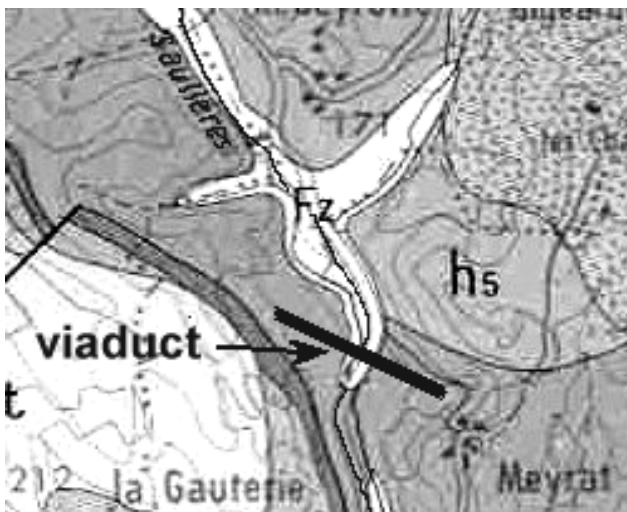


Figure 2. Localization of the projected viaduct on the geological map (1/25000): Fz = alluvium, h5 = sandstone.

The viaduct of Saulières will be 30 meters high and 450 meters long. It is designed with an ENE / WSW trend to cross a valley and a local interest road. It will be constructed to be a part of a bypass around the regional city of Brive (fig. 2).

The piles of this bridge will be constructed within the sandstone formation of the Permian basin of

Brive, away from the principal NE-SW faults. The dip of the two opposed slopes is respectively 30 to 40 degrees (West side) and 50 to 60 degrees (East side). At the surface, the bottom of the valley is made of alluviums whose thickness exceeds 4 meters, and the soil on the slopes is made of colluviums, whose thickness varies from 0.3 to 5 meters. These deposits are lying on sandstones which are often altered at the top and affected by intense fracturing in various directions.

## 2.3 Design of the foundations of the viaduct

For design and construction purposes of foundations on rock, the current methods consist to estimate the bearing capacity on the basis of an empiric classification of rocks mainly according to the rock matrix properties (Bieniawski, 1976). Still, some recommendations of building codes for this application mention the existence of fractures in the rock mass, but generally the estimation is obtained from continuum mechanics. For instance, the approach used for the viaduct of Millau (France) is based on a classification of rocks from field data, laboratory tests and drill core description (Rat and Givet, 2000). This approach is global and allows for estimating the mechanical characteristics as deformability and resistance at the scale of the rock volume involved. In the recent years, because of an increase in applied loads in engineering practice, and taking into account seismic alea and other major natural hazards, the designers of road infrastructures in rock masses pay more and more attention to the role of fractures in the stability computations of construction works (Maurin, 2001).

For the viaduct of Saulières, the bearing capacity,  $Q_{pu}$ , and the ultimate shaft resistance,  $Q_{su}$ , are calculated on the basis of pressuremeter or dilatometer tests. The piled foundations drilled will be anchored in the compact sandstones. The load on each foundation, is evaluated according to the number of piles under consideration for viaduct, about  $1.9 \cdot 10^5 N$  by pile, and  $0.5 \cdot 10^5 N$  for the two abutments. These values are tested in the numerical study following.

## 3 A CONTINUOUS REPRESENTATION

### 3.1 A continuous 3D modeling (finite elements)

A model was realized in 3D using the finite element code CESAR-LCPC. The topography is roughly drawn and it gives a simplified image of the load repartition at the bottom of the viaduct piles. The mesh resolution is enhanced in the neighborhood of the piles (fig. 3). But the piles are not explicitly represented with this software.

The load is imposed at a 10 meters depth, i.e. at the top level of the intact sandstone layer which sup-

ports the piles. The vertical stress increases according to the empirically estimated load on each pile.

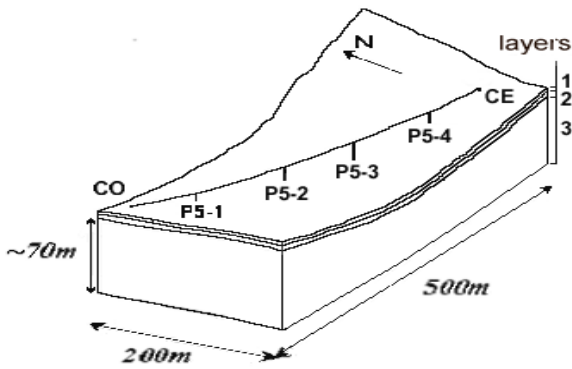


Figure 3. A 3D stratified model of the Saulières valley with the position of the viaduct, piers, and drillings CO (West) and CE (East).

The mechanical model is composed of three superposed layers with different constitutive materials, each one being given an elastic-plastic behavior with Mohr-Coulomb criterion. The characteristics of materials follow:

- > layer 1:  $E = 1 \text{ GPa}$  ;  $\nu = 0.25$  ;  $C = 1.E4 \text{ Pa}$   
 $\phi = 22 \text{ degrees}$ ,  $\psi = 22 \text{ degrees}$
- > layer 2:  $E = 5 \text{ GPa}$  ;  $\nu = 0.25$  ;  $C = 1.E5 \text{ Pa}$   
 $\phi = 25 \text{ degrees}$ ,  $\psi = 25 \text{ degrees}$
- > layer 3:  $E = 50 \text{ GPa}$  ;  $\nu = 0.35$  ;  $C = 1.E7 \text{ Pa}$   
 $\phi = 35 \text{ degrees}$ ,  $\psi = 35 \text{ degrees}$

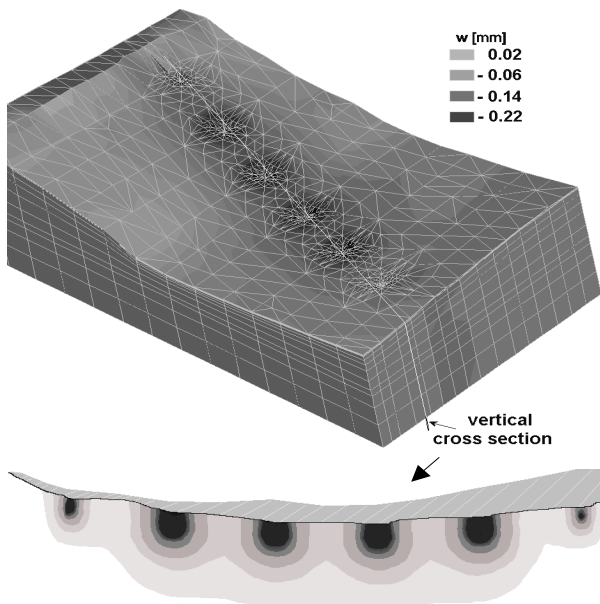


Figure 4. Vertical deformations after punctual loading and 2D vertical cross-section (computed with CESAR-LCPC).

Layers 1 and 2 are both discretized into two mesh strata, and layer 3 has seven mesh strata with increasing thickness along depth. The values of the characteristics of intact and altered sandstones have been estimated including a secondary alteration due to fractures, and the values for the characteristics of colluviums are reduced down to a point imposed by the resolution of our finite element computations.

After loading, some deformation appears in the soil at the bottom of the piles (fig. 4). Deformation is centered on the points where the load is applied, and propagates through the layers in a quite symmetrical way. Notice that the displacements are insignificant since they are found to be in the order of 0.02 mm at the surface and 0.20 mm at the loading point. When the plastic state is attained in the superficial layer, the computations are soon interrupted for non-convergence. For this reason, it was not possible to reduce the mechanical properties of colluviums in order to increase the displacements.

### 3.2 A continuous 2D modeling (finite differences)

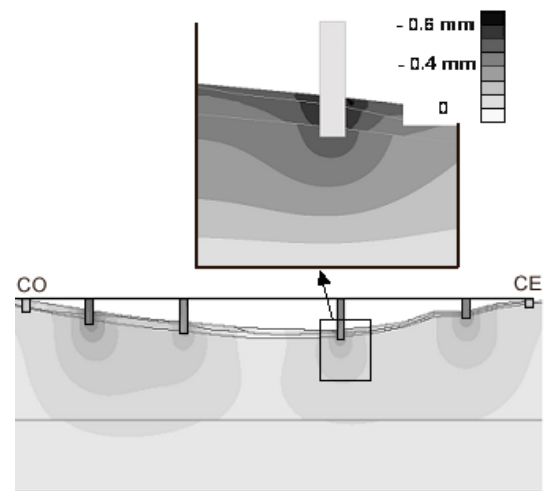


Figure 5. Vertical deformation after loading at the top of the pile, computations with UDEC.

On the basis of the geometry of the geological layers, a computation is performed by finite differences with the UDEC code, at the scale of a single pile in a vertical longitudinal cross-section. The piles of the viaduct are explicitly represented in this software. The bottom of the pile is fixed at the top level of the intact sandstone layer.

We use a transfer factor  $\mu$  which is recommended for estimating 2D equivalent stresses from the 3D load applied at the top of a pile with a square base (Rachez, 1997):

$$\mu = \frac{4BH + B^2}{2H + B} = 11$$

where the height of the pile is  $H = 30$  meters, and the edge length of the square base is  $B = 6$  meters.

After we have deduced the settlement at the interfaces between the layers, we can note on figure 5 that the maximum displacement is close to the result of the 3D model with CESAR-LCPC. A close-up on the pile P3 with a better mesh shows locally a greater settlement around the bottom of the pile, progressively decreasing from a layer to the next with increasing depth.

## 4 A DISCONTINUOUS REPRESENTATION

### 4.1 The failure mechanisms of foundations on rock

Loading in continuous models gives a global idea of the intensity of deformation within the soil, but does not make it possible to detect the different orientations of the displacements which arise from the natural stratification of the rocks. For different types of the foundation and of the structure of the rock under loading, different mechanisms of potential instability leading to the failure were considered, from geology conditions (i.e. wedge, toppling or plane failure) or shear and deformation of fractured rock mass due to compression (fig.6) (Wyllie, 1992). It is thus very important to improve our knowledge of the rock mass in order to define the better type of foundation in the studied case. To make numerical modeling realistic enough, we must face the difficulty to reproduce the fracture network from punctual data.

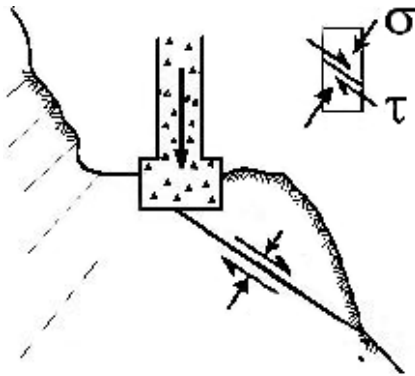


Figure 6. An example of potential mechanism of failure of a fractured rock mass under loading.

### 4.2 Characterization of the rock slope

From field prospecting and aerial photograph analysis we have a good knowledge of the main discontinuities, faults, and folds. Interpretation of well logging and borehole data results in a structural model which takes into account the sedimentary layers and the fractures. However, these data are very expensive to obtain and they are available only in a limited number. Also, the interpolation of data between several boreholes is very difficult. In this project, two boreholes have been realized and they give a 30 meters cumulate length of cores. The first was done at the East end of the projected bridge (CE), the other at the West end (CO). These boreholes are supplemented by 22 pressuremeter profiles (Chahine et Nagel, 2003).

An example of a stereographical treatment of data is shown on figure 7a. We identify two fracture sets that are characteristic of borehole CO, near the West end. Within 13 meters of cores, 50 discontinuities have been detected in the depth interval from 2 to 10

meters. The mean dip vector (azimuth and plunge angles) is (212-43°) for set S1 and (58-56°) for set S2. Respectively the mean spacing is 0.24 m and 0.30 m. We do not use the data of the drilling carried out at the end CE because of a low quality in numerical imaging and because the discontinuities have very sparse directions. Thus, the statistical analysis of the data from this borehole is difficult to interpret.

In order to extrapolate the fracture network to a volume larger than that recognized by drilling, we use data-processing tools which allow the stochastic generation of discontinuous 3D models statistically equivalent to real borehole data. The FRACA software, Beicip (IFP) Company, was initially conceived for oil research (Cacas and al., 2001) but can be adapted for civil engineering problems.

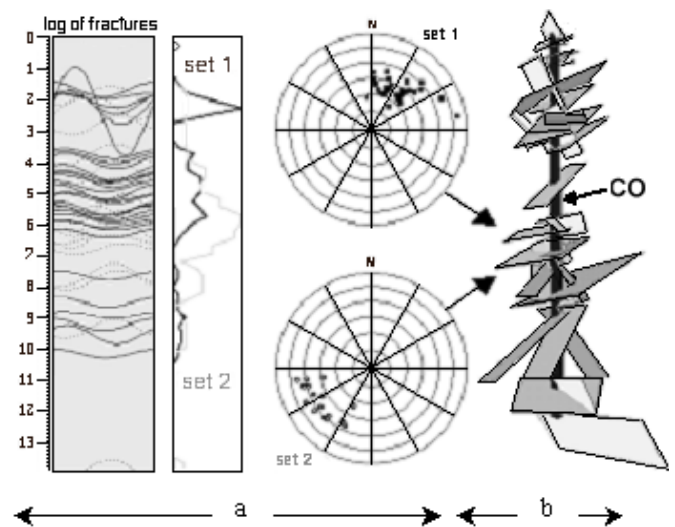


Figure 7.

- a) Stereographic representation of the normal vectors of fractures, borehole CO (lower hemisphere, ISRM convention)
- b) intersection of the borehole with simulated rectangles standing for the fractures of the two identified sets (FRACA)

By introducing an empiric value of 5 meters for the average size of the fractures, we simulate a series of virtual boreholes to explore the variability of the spatial distribution of our virtual network. An example of discontinuities connected to borehole CO is displayed in figure 7b.

### 4.3 Numerical modeling by the distinct element method

From the rectangles generated in the neighborhood of the abutments and foundations of the piles, we extract the 2D co-ordinates of their intersections with a cross-section (fig.8). Up to now, we do not have any numerical tool, even in 2D, capable of treating such a complex network of fracture traces. So we choose to run UDEC with oversimplified schemes. To remain within the limits of data-processing resolution of the computer code UDEC, the representation of a

cross-section of a discontinuous model over the entire length of the viaduct would imply a considerable reduction in precision. We thus will position our cross-section at the scale of the pile n°3 of the viaduct which is supposed to support the most significant effort.

*stochastic generation of discontinuities  
(3D model from FRACA)*

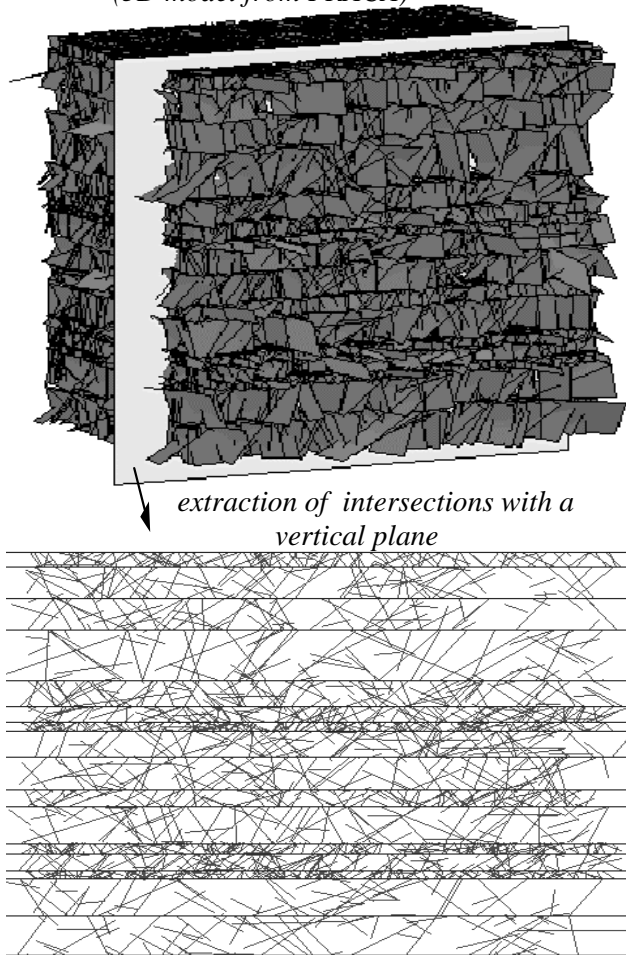


Figure 8. Trace intersections between a vertical plane and the rectangles representing the fractures.

Moreover, the fractured part is reduced to a zone under the pile. Lastly, we tune the distribution of the two fracture sets and the value of mean dip angle of dip (for one set) in order to show the influence of the geometrical uncertainty.

Values equivalent to those of preceding calculations were then applied to constitutive materials. A friction angle of 20 degrees and a stiffness of  $10^{10}$  N.m for the joints of the fractured zone.

The first case represents two sets of discontinuities with trace persistence 1 meter, respectively with an apparent dip of 58 degrees and 115 degrees (fig. 9a). Under the effect of the load applied at the top of the pile, the deformations are propagated in the rock mass in an asymmetrical way, with maximal amplitude lower than 2 millimeters. Notice that the distri-

bution of the iso-values of displacement follows the dips of the two discontinuity sets.

The second case represents the two same sets but their persistence follows a uniform distribution in a close range of mean value 1 meter (fig. 9b). In this case, the most highly deformed zone is tightened, the amplitude at the bottom of the pile increases slightly and the limit between the iso-values of displacement follows the traces of some discontinuities.

The third case represents the two sets with random persistence but with stronger dips: 72 degrees and 115 degrees (fig. 9c). The pile of the viaduct sinks like a punch, while following the dips of the two sets: the maximum value of displacement at the bottom of the pile approaches 2,5 millimeters.

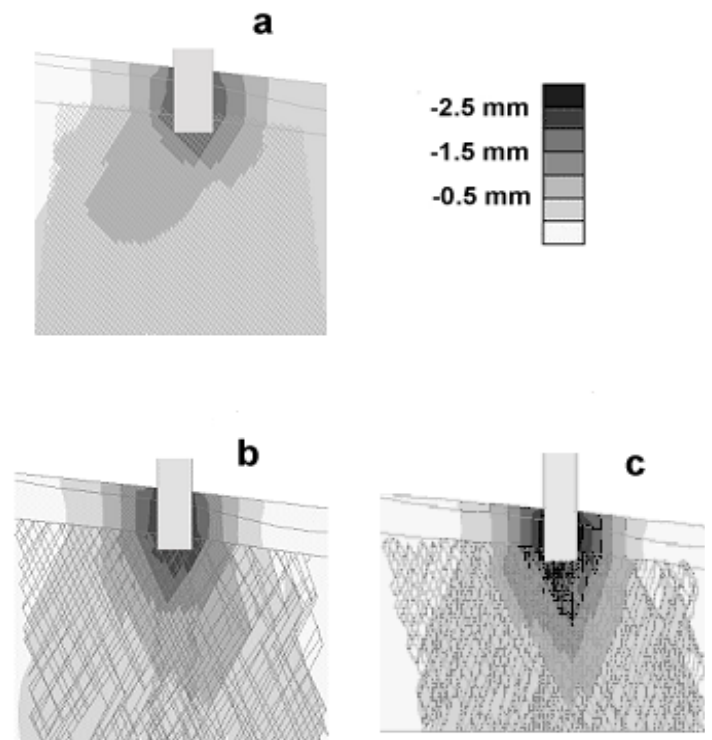


Figure 9. Vertical displacements due to loading on pile P5-3 for three discontinuous models, computations with UDEC.

## 5 DISCUSSION

The difficulties encountered when modeling rock slopes are well known: the constitutive materials are diverse, the geological survey is not always adapted to the specificity of the project, and the fractures are partially detected (Fouché and Diebolt, 2004). It appears that the dimensioning of the works rather rises from a know-how informed by databases and practice. This pragmatic approach has proved to be reliable in the construction industry. It is generally supplemented by a numerical approach using the mechanics of the continuous media. In this paper we have shown that, in the case of loading on the piles of a viaduct, this method gives some useful indications on the spatial repartition of the ground defor-

mation. It can be improved by resorting to an homogenization method, which includes the discontinuous character of the rock into an equivalent continuous medium (Sulem and Cerrolaza, 2001).

But the indications obtained by the former methodology are essentially qualitative and do not allow to work without over-design. If one wishes to adapt the dimension of the foundations to the nature of the rock, we need some quantitative estimate of the displacements and the threshold of failure. Then, we have to explicitly represent the geometry and mechanics of the discontinuous media.

The stages of this kind of study are multiple, from in situ data collecting to numerical simulations. Starting from a minimum of geological and geometrical statements, the stochastic reconstitution of the fracture network makes it possible to limit the number of possible cases of simulations. Then it is necessary to carry out a parametric study of the characteristics of constitutive materials of the rocks and joints to qualify the potential state of failure at the bottom of the foundations. This approach is made possible by the distinct element modeling (Cundall and Hart, 1993).

If 2D modeling is rather easy to use, it however requires considering cross-sections perpendicular to the main fracture set of the rock mass to be a reliable representation of reality. The 3D structure is, on the other hand, rather delicate to work out, with a more restricted number of distinct elements, but it opens more prospects for the study of the stability of the fractured rock volume.

## 6 CONCLUSION

Based on the geological analysis of the area, the reconstitution of the fracture distribution over the length of a bridge is possible from borehole data by using a specific modeling tool. The extraction of virtual boreholes at the position of the foundations then allows the construction of 3D or 2D models that turn out to be more rigorous and more complex than those resulting from the surface survey.

The numerical study was limited to particular cases of fracture densities which shows that, at the scale of a pile, the homogenization of the media seems satisfactory.

Nevertheless, in spite of the reduced dimensions of our volume of interest and of the dispersion of the fracturing of the rock, numerical computations of 2D discontinuous models show a strong anisotropy in the distribution of the iso values of displacement, with maximum amplitude locally higher than the values given by the continuous models. While approaching the natural configuration of the rocks, numerical modeling can then highlight the preferential orientations of the stresses or the deformations and contribute to detect the potential risks of failure.

*We are grateful to the Beicip (IFP) Company and the Regional Laboratory of Bridges & Roads of Clermont-Ferrand for their invaluable collaboration which enabled us to carry out this first testing of our methodology.*

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