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# Statistical reconstitution of fracture networks for numerical studies in geotechnics

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**ABSTRACT:** For design purposes of foundations on rock, the current method consist to estimate the bearing capacity on the basis of an empiric classification of rocks. The internal structure of a rock mass is hidden, which makes difficult to identify the relevant class at depth. To deal with this difficulty, we use some stochastic algorithms to simulate many realisations of a fracture network conditioned by measured information. Then we apply two complementary numerical methods to give an estimate of the deformation to occur around the piles of a viaduct projected on rock slope. We present the case of the viaduct of Saulières (France) with two levels of analysis depending on scale. In a volume of rock large enough where the multiple order discontinuities do not favour any potential failure surface, we can represent the rock mass as an equivalent continuum (section 2). Using continuous numerical modelling (here finite elements) at the viaduct scale, we compute the repartition of the deformation under the piles within the rock mass. At a specified local scale, another method is to represent the rock mass as a system of homogeneous blocks (section 3). Then, using discontinuous modelling (distinct elements), we focus on local strain around a pile.

## 1 Foundation on a rock slope

### 1.1 Design of foundations according with scale of fractures

The stability of rock slopes generally appears to be sufficient for building roads and common infrastructures. But the internal equilibrium within a rock mass can be modified by high level loading due to greater structures combined with natural weakness due to heterogeneous and discontinuous formation of the geological medium. A foundation must be designed so that the resultant load exerted on it does not cause any lateral sliding of the foundation base, ground collapse underneath the structure, intolerable settlement or tilting 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. 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 constitutive materials as well as the properties of joints are likely to influence the mechanical behaviour of the rock mass. 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, according to the rock matrix properties and the state and spacing of the discontinuities at the sample scale (Bieniawski, 1976). Some recommendations of building codes for foundations do indeed mention the role of fractures in rock masses. But the bearing capacity is generally obtained from continuum mechanics. The rock characteristics as deformability and resistance are estimated at an intermediate scale such that the fracture network can be homogenised. This approach was followed in the case of the viaduct of Millau (France) achieved in 2004, with an evaluation of the rocks from field data, laboratory tests and drill cores (Rat and Givet, 2000).

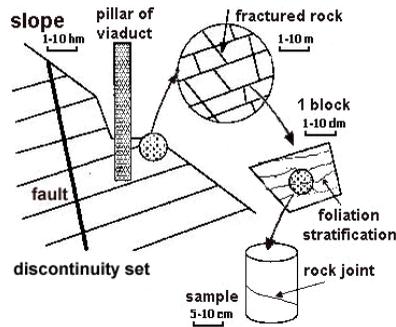


Figure 1. The rock mass at different scales.

In order to determine the limit load for foundation on rock, estimates resulting from empirical studies are also proposed using the nonlinear criterion of failure from Hoek and Brown (Hoek and Brown, 1980). This method, supplemented by theoretical assumptions, was developed into specific procedures of application for various types of rocks, depending on their state of fracturing (Serrano and Ollala, 1996). 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 construction works (Maurin, 2001). In this paper, we continue the study introduced in Alfonsi and Fouché (2003).

## 1.2 Case study: the rock-founded viaduct of Saulières (France)

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

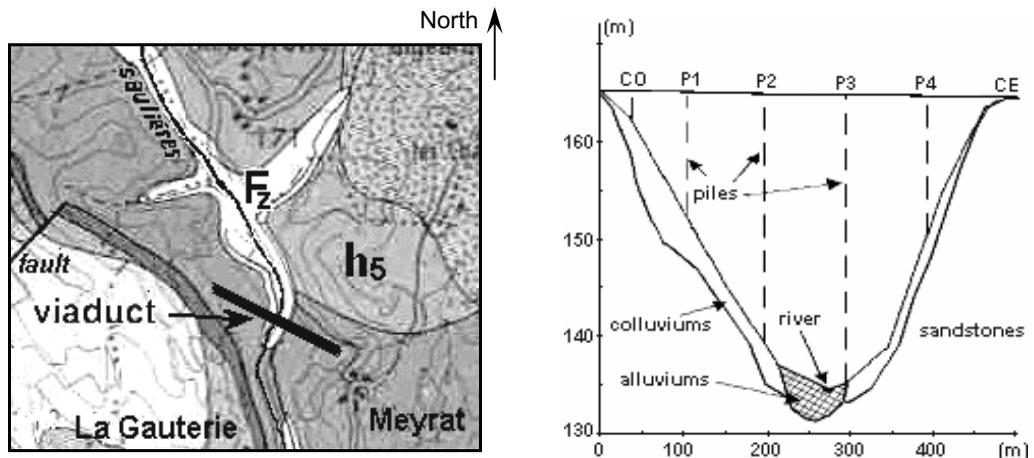


Figure 2. Localization of the projected viaduct on the geological map (1/25000): Fz = alluvium, h5 = sandstone. Vertical WNW-ESE cross-section of the valley, position of piles and platform.

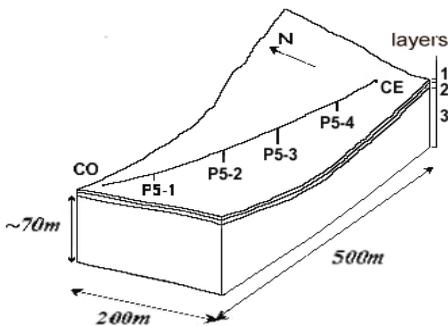
The piles of this bridge will be constructed within the sandstone formation of the Permian basin of

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. The pile foundations will be anchored within the compact sandstones.

For this viaduct, the bearing capacity  $Q_{pu}$  and the ultimate shaft resistance  $Q_{su}$  have been calculated on the basis of pressuremeter or dilatometer tests. The load on each foundation is evaluated according to the number of piles under consideration, about 1900 kN by pile, and 500 kN for the two abutments. These values are tested in the following numerical study.

## 2 Continuous 3D modelling at the viaduct scale

A 3D model was first realised using the finite element code CESAR-LCPC which allow us to draw the topography. The piles are not explicitly represented but the mesh resolution is enhanced in their neighborhood (fig. 3). 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. Layers 1 and 2 are both discretised into two mesh strata, and layer 3 into seven mesh strata with increasing thickness along depth. The values for the mechanical characteristics of intact and altered sandstones have been estimated accounting for a secondary alteration due to fractures, and the values for the characteristics of colluviums are reduced as much as allowed by the resolution of our finite element computations.



The characteristics of materials are:

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

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

After loading, some deformation appears in the rock at the bottom of the piles (fig. 4). The iso-deformation curves are centered on the horizontal segments where the load is applied, and propagate quite symmetrically through the layers. Let notice that the values of displacement in figure 4 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 segments. This is mostly due to lack of resolution of the mesh which made not possible to attain the design load, but only about 75% of it. Moreover, because of the mechanical contrast between layers 1 and 2 and the sandstone layer 3, the plastic state is quickly reached in the superficial layers.

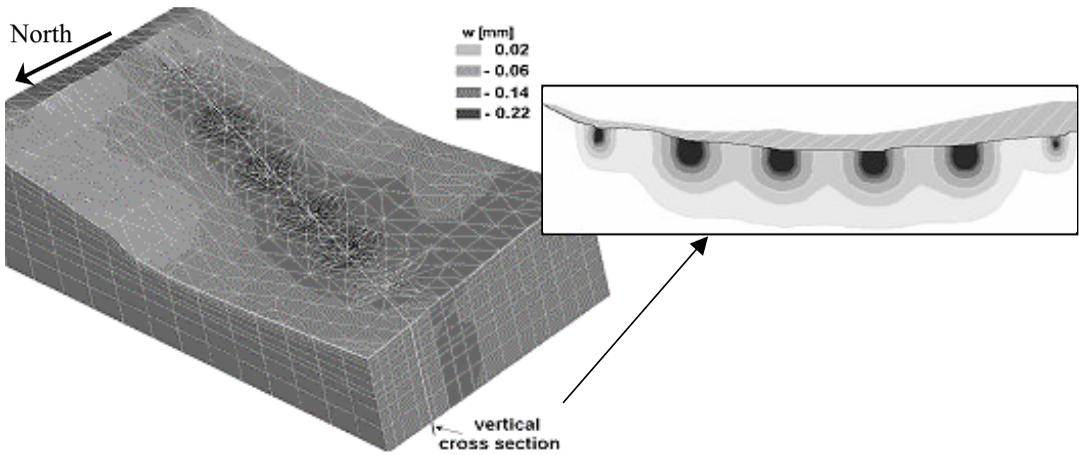


Figure 4. Vertical deformations after punctual loading and 2D vertical cross-section (computed with CESAR-LCPC).

At the scale of the viaduct indeed, trying both to honor the complex geometry of the field surface and to represent the 3D deformation in contrasted layers, we encountered some numerical problems with a too high number of mesh elements. We obviously could further simplify the geometrical and geological model in order to succeed in a complete computation. But this threaten to become unrealistic. We alternatively could reduce the scale of the model down to one or two piles. But at this scale, homogenisation of the fracture network would be abusive and we advocate for a discontinuous model.

Consequently, concerning the ultimate bearing capacity of the foundations on the sandstone layer, this method used at the scale of the viaduct does not allow to conclude. Nevertheless, it gives an insight into the displacement repartition at the bottom of the viaduct piles. It is possible to observe how the deformation around a pile could interfere with another. This kind of modelling can be helpful for decision regarding the number and size of the piles.

### 3 Discontinuous 3D-2D modelling at the foundation scale

#### 3.1 Failure mechanisms of foundations on a rock slope

Continuous models give a global idea of the intensity of deformation within the rock mass, but does not allow to detect the different orientations of the displacements which arise from the natural discontinuities of the rocks. For different types of foundations and natural architectures of the rock under loading, different mechanisms of potential instability leading to the failure can be observed (i.e. wedge, toppling or plane failure). It is thus important to improve our knowledge of the rock mass in order to define the better type of foundation in the studied case (Wyllie, 1992).

This approach is implemented through distinct element modelling with UDEC (Cundall and Hart, 1993). As shown in the following study, the quality and fidelity of the fracture network simulation has a significant influence on the numerical results. Same values as preceding calculations are applied to constitutive materials. We apply a transfer factor  $\mu$  which is recommended in literature for estimating the 2D load  $F_{2D}$  equivalent to the 3D load  $F_{3D}$  applied at the top of a pile with a square base (Rachez, 1997):

$$\mu = \frac{4BH + B^2}{2H + B} = 11 \quad \text{with} \quad F_{2D} = \frac{F_{3D}}{\mu}$$

where the pile's height H is 30 meters, and the edge length B of the square base is 6 meters. A

friction angle of 20degrees and a  $10^4$  MN.m stiffness are chosen for the joints of the fractured zone.

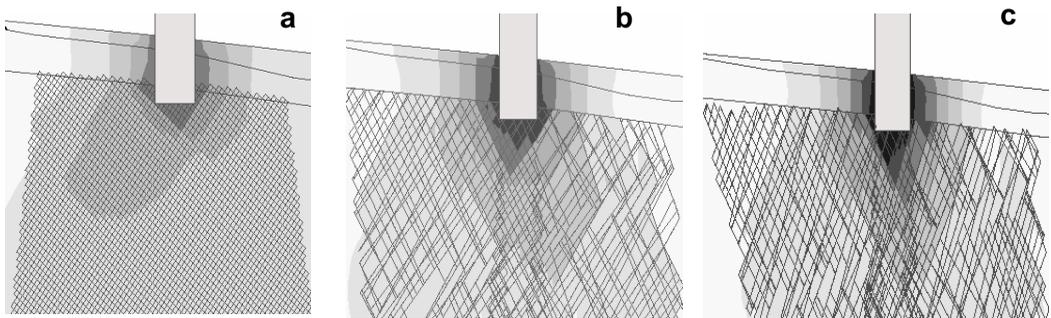


Figure 5. Vertical displacements due to loading on pile P5-3 for three discontinuous models, computed with UDEC.

The first case represents two sets of persistent discontinuities with constant trace spacing 1 m, respectively with an apparent dip of 58 (to east) and 65 (to west) degrees (fig. 5a). Under the effect of the load applied at the top of the pile, we have the classical result that the deformations propagate asymmetrically in the rock mass. The distribution of the iso values of displacement follows the dips of the two discontinuity sets. Here the maximal amplitude is lower than 2 mm.

The second case represents the same two sets but their spacing follows a uniform distribution in a close range of mean value 1 m (fig. 5b). 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 spacing but the west-dipping set has a stronger dip of 72degrees (fig. 5c). The maximum value of displacement at the bottom of the pile approaches 2.5 mm.

The failure mechanism appears clearly in case 2 and especially in case 3: the pile sinks like a punch, while following the dips of the two sets. This was less visible in the reference case 1. Here is underlined the importance of a precise definition of the fracture network to predict a potential instability.

If 2D modelling is rather easy to use, it however requires considering cross-sections perpendicular to the main fracture sets of the rock mass to be a reliable representation of reality. Then it allows to carry out a parametric study of the rock constitutive materials and the joints to qualify the potential state of failure at the bottom of the foundations. On the other hand, the 3D structure opens more prospects for the stability study of the fractured rock volume but it is rather delicate to work out, with a more restricted number of distinct elements. We propose a half-way method where the 2D model will not be an a priori simplified scheme but an oriented cross-section through a 3D geometrical model of the fracture network.

### 3.2 Geometrical simulation of a stratified 3D fracture network

The difficulties encountered when modelling rock slopes are well known: the constitutive materials are heterogeneous, the geological survey is not always adapted to the specificity of the project, and the fractures are partially detected (Fouché et al., 2002). To make numerical modelling realistic enough, we must face the difficulty to reproduce the 3D fracture network from local or point data (Fouché and Diebolt, 2004).

From field prospecting and aerial photograph analysis we have a good knowledge of the main discontinuities, faults, and folds. Tools of structural geology also allow identifying the main fracture sets. Then, interpretation of well logging and borehole data contribute to a structural model which takes into account the sedimentary layers and the fractures. Further information on the nature and

fractures of the rock mass is given by cores. But these data are very local and the real fracture size and shape remain unknown (Fouché et al., 2001). Moreover, they are very expensive to obtain and thus available only in a limited number. Also, the interpolation of data between several boreholes is a difficult task.

To deal with this difficulty, we use stochastic algorithms to simulate many realizations of a fracture network conditioned by measured information. The computing tool FRACA used in this study was designed by the Beicip Company of the French Institute for Petroleum (IFP). This software, initially conceived for oil research (Cacas and al., 2001), can be adapted for civil engineering problems. The fracture network is constrained by a model of the geological medium which is stratified and heterogeneous. From the spatial coordinates of the intersections between fractures, we reconstitute a 3D system of rock blocks.

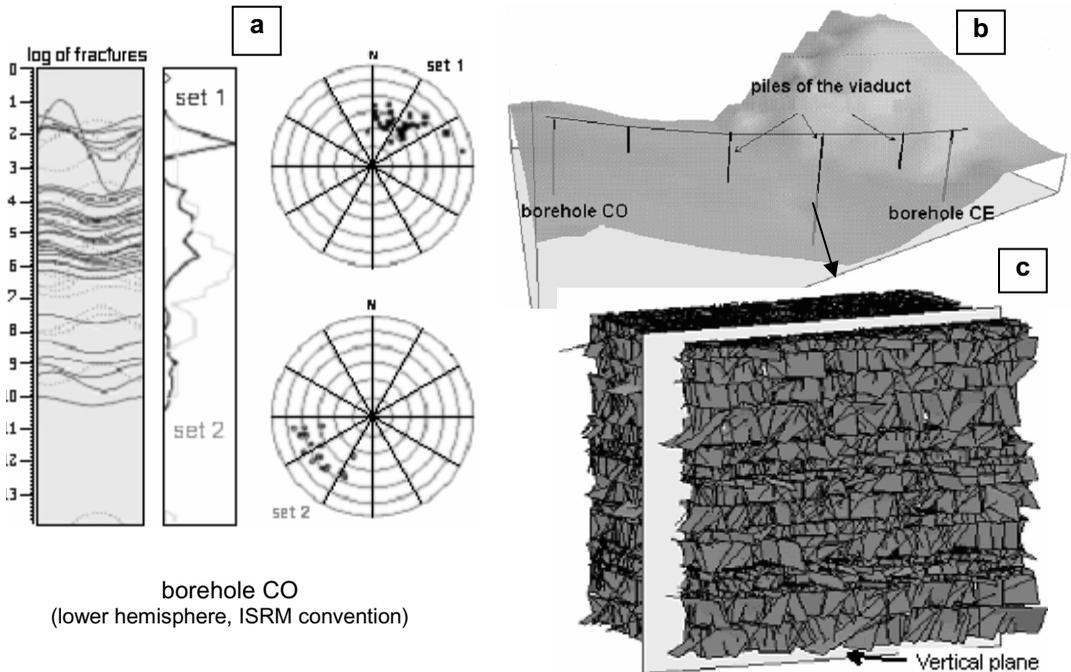


Figure 6. a) Stereographic representation of the normal vectors of fractures. b) Insertion of the results of the stereographic analysis into a FRACA geological structural model. c) Extraction of a 3D cell under the pile P5-3.

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). First we made some stereographical treatment of CO data (fig. 6a) and we identify two fracture sets that are characteristic of borehole CO. Within 13 meters of cores, 50 discontinuities have been detected in the depth interval from 2 to 10 meters. By introducing an empiric value for the average size of the fractures, we simulate some virtual boreholes to explore the spatial distribution of our virtual network (fig. 6b). In order to extrapolate the fracture network to a volume larger than that recognized by drillings, we use data-processing tools which allow the stochastic generation of 3D discontinuity networks statistically equivalent to real borehole data (fig. 6c).

### 3.3 Deformation in oriented-stratified-fractured cross-sections

At this point, we use a straightforward 3D to 2D transition. From the rectangles generated in a 3D cell, we extract the 2D co-ordinates of their intersections with a vertical cross-section. This operation produces a 2D network of fracture traces. We can make such vertical cross-sections in several directions. Then the UDEC code constrains the trace network to be fully connected i.e. without dead ends. The persistence of each trace is slightly modified to satisfy this condition. In our geological model, the thin strata are densely fractured. Another modification is to replace each one of these strata by a layer made of a weak continuous material. Figure 7 displays iso-values of vertical displacement in the rock mass under increasing load on the top of the pile.

Scheme **7a**: stratification of the rock mass with variable thickness of strata; symmetrical deformation occurs in the rock with a maximal value of 4 mm in the vicinity of the pile; thinnest weakest strata absorb the deformation, which results in discontinuous iso-curves of displacement.

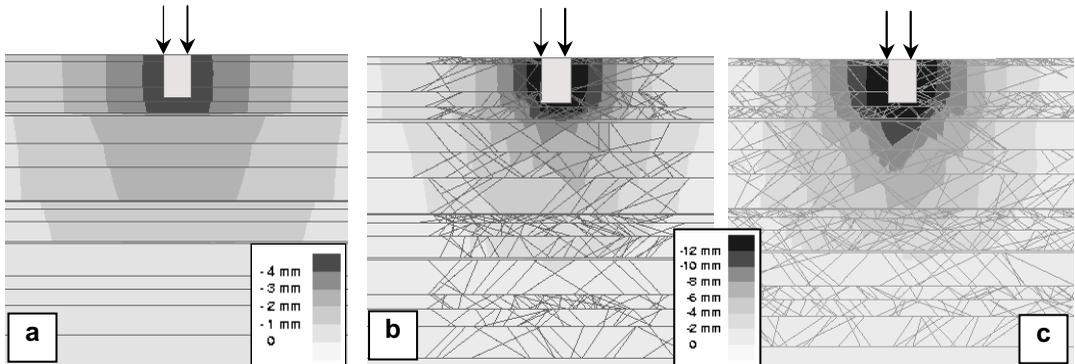


Figure 7. Vertical displacements due to loading on pile P5-3 for three layered models, without fractures (a), with 500 fractures (b), and with 750 fractures (c); computations with UDEC.

Scheme **7b**: the 3D fracture network is sampled by the cross-section; deformation, three times higher than the unfractured scheme, is concentrated around the pile and rather along high-dipping fractures; the iso-deformation curves propagate along the fractures through the layers and absorption by weak strata is minimised.

Scheme **7c**: with increasing the number of fracture traces, dispersion of the iso-value curves is increased in area and less controlled by fractures; it tends to look like scheme **a** but with deformation values similar to scheme **b**.

In this configuration, we do not take into account the slope of the rock mass. It is to be done in further developments.

## 4 Conclusion

The stages of this study are multiple, from in situ data collecting to numerical simulations. 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 specific modelling tools. The extraction of virtual boreholes allows to control the construction of the 3D model which is more complex and more rigorous than a model resulting only from the surface survey. Starting from a minimum of geological and geometrical statements, the stochastic reconstitution of the fracture network makes it possible to explore the possible cases and chose some realistic simulations.

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 would be satisfactory in some cases. The study

could then be continued by resorting to an homogenization method including the discontinuous character of the rock into an equivalent continuous medium (Sulem and Cerrolaza, 2001). In spite of the reduced dimensions of our rock volume and of the fracture dispersion within the rock mass, 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 modelling can then highlight the preferential orientations of the stresses or the deformations and contribute to detect the potential risks of failure. 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 deformation.

## 5 Acknowledgements

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