Improving rock mechanics education by using case studies analyses

D. Stematiu
Technical University of Civil Engineering Bucharest, Romania

ABSTRACT: Rock mechanics is part of civil engineering curricula. The objectives of the course are to present clearly and concisely the basic knowledge of rock mechanics implied in civil engineering profession. The course is strongly oriented towards the engineering practice. All the details are kept to a minimum and the subjects are presented at a level appropriate for students. The objectives can be partly achieved if and only if a sufficient number of lectures are dedicated to demonstrate the practical applications of these principles by examples. The presentations of a selected number of case studies allows for a better understanding of the large variety of possible situations. The paper presents some of the concepts that need to be underlined by case studies. The case studies are dedicated to past technical accidents in order to raise student interest.

1 INTRODUCTION

The topic of the rock mechanics course may be summarized as follows: (1) the rock mass classification, (2) the rock strength and rock mass deformability (3) rock permeability and its improvement through grouting and drainage for both rock foundation and underground excavations, (4) rock slope stability and the required stabilization constructive measures (5) underground excavation stability and support systems for tunnels and large caverns.

Some of the subjects are directly connected to the previous knowledge in geotechnics and hydraulics and can be easily assimilated. However, the discipline presents a certain difficulty for students asking for a new way of thinking. The thing that authorizes us to speak of a new discipline is that rock mechanics attempts a mechanical synthesis, which has already been successful in a number of fields, but for a material with a behavior mainly dependent on the discontinuities. In this way it differs from the common approach and errors are made by students because the rock is sometimes improperly considered similar to other materials with which they are more familiar, e.g., continuous materials.

2 UNDERSTANDING ROCK MASS DEFORMABILITY

2.1 *The problem*

It is essential to make the distinction between internal deformation within the rock mass and surface movements. It is the first category that allows the engineer to understand the intrinsic behavior patterns of the foundation, whereas the second is adequate for analyzing the reactions just under the base of the civil structure. For a very long time, only surface deformation was considered, in other words, the foundation was conceived as an imaginary equivalent continuum whose surface movements would be the same as those of the actual foundation.

The internal deformations can only be found by taking into consideration the actual discontinuous medium or at least a model of it. We are now well aware that such deformations differ considerably from those of a continuum when the confining stresses are small. The discontinuous model is more relevant to surface workings such as the foundations for structures than for underground workings.

The students have to understand that because of the irreversible effects of closure of the cracks and fissures the deformation pattern may strongly modify the foundation – structure interaction with severe consequences. Any serious study of the actual deformations of the rock at depth calls for the use of mathematical models. A case study dedicated to Poiana Rusca dam illustrates to students how the main faults control the deformability of the abutment and the change of the design imposed by this particular situation.

2.2 *Changes of the design of an arch dam imposed by uneven pattern of foundation deformation.*

Poiana Rusca arch dam, with a total height of 95 m, is under operation on Hideg River in Romania in order to provide storage for water power production. The dam body is rather thick, with 5 m at the crest and 26 m at the foundation level (Fig. 1).

The dam foundation area is affected by a well developed system of faults (Fig. 2). The major faults (F) are rather continuous, exhibit a significant opening and their lateral zones, with squeezed and altered rock, extend over 2 to 8 m. On the left bank three major faults (Fl, F2 and F9) cross the dam foundation surface in the mid-third of the dam height.

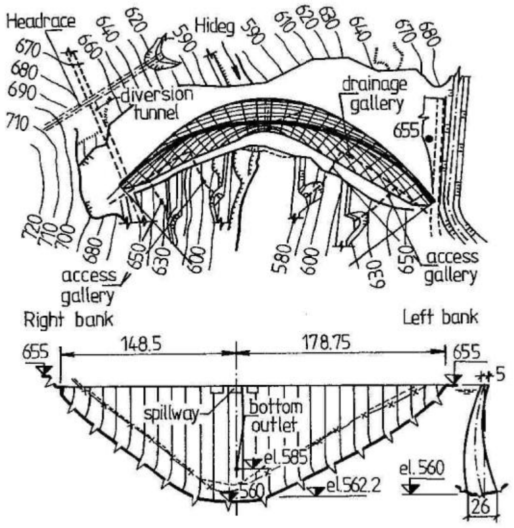


Figure 1. Poiana Rusca dam: plan view and longitudinal profile.

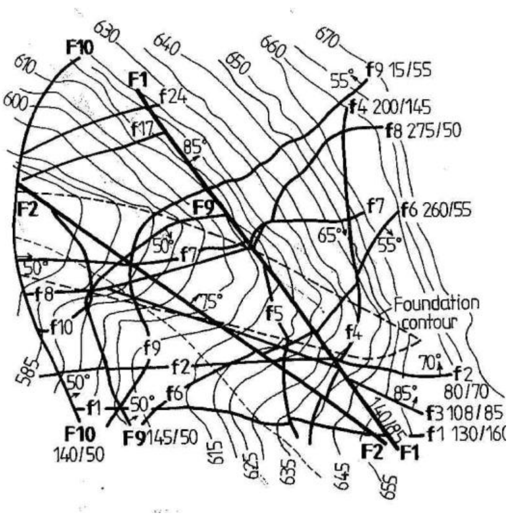


Figure 2. Site geology of the left bank of Poiana Rusca dam.

A more evident image of the faults interference with the dam foundation may be seen in Figure 3.

At the design stage the foundation rock was treated as an equivalent elastic body, the local particularities of the rock mass being either disregarded or taken into account by appropriate changes of the elastic moduli.

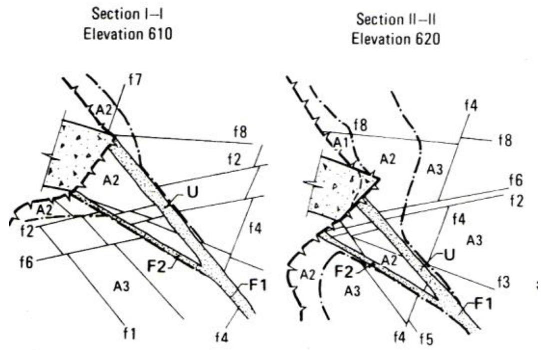


Figure 3. Dam foundation on mid third of the left abutment.

However, during the excavation work it was revealed that dam-foundation interaction on the left bank is significantly affected by the major faults Fl and F2, and particularly by the very deformable gouge of fault Fl. Consequently, a new dam-foundation interaction analysis was required, but this time only for the affected zone.

This analysis was carried out on the basis of three 2D-plane strain finite element models, corresponding to the characteristic horizontal sections shown in Figure 3. One of the three models, corresponding to the horizontal section at el. 610 m is shown in Figure 4a. Faults Fl and F2 were individually discretized and the boundaries between rock zones Al to A3 (see Fig. 3) were included in the FE mesh, allowing for the appropriate input of the rock elastic constants. The arch stresses at the limit of the discretized dam sections were used as external distributed loads. The behavior of the rock and concrete was assumed to be linearly elastic. Some of the results obtained for the horizontal section at el. 610 m are shown in Figure 4. The displacement pattern (Fig. 4b) shows that the rock triangle between faults Fl and F2 has almost rigid body movement, the main strains being induced in the filling material of the faults.

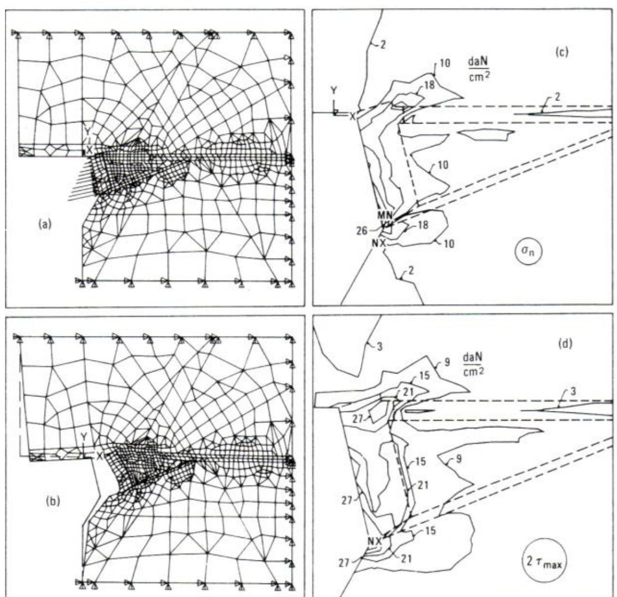


Figure 4. Finite element model and the main results.

The 2D analysis showed the large displacement tendency of at least one rock block induced by the deformability of the fault gouge. The mobilized shear was significantly larger than the shear strength due to unfavorable behavior induced by fault Fl.

Internal movement of some rock blocks and tangential displacements could be induced by the dam thrust as a result of the considerable deformability of the fault gouge. Consequently, the most obvious measure is to replace the material in the fault with concrete. Alternative solutions could be used, for example, replacing only certain zones, the main idea being to provide a rigid connection between the fault faces across its opening.

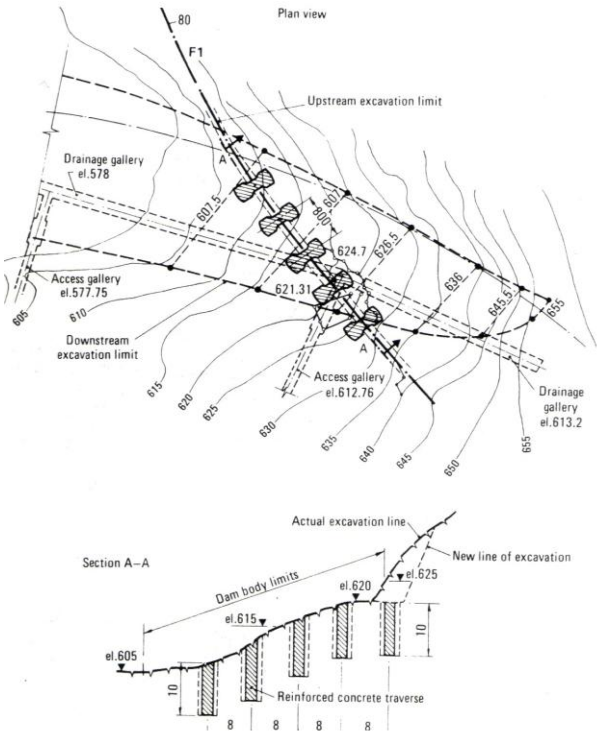


Figure 5. Fault consolidation by concrete traverses.

The proposed alternative for foundation strengthening is presented in Figure 5.The consolidation measure consists of providing several reinforced concrete cross-walls ('traverses') to 'seam' fault Fl, located in the dam foundation area. After a review of the investment cost increase, the owner has decided to avoid the unfavorable foundation conditions by changing the dam height such as the new footprint of the dam will be outside of the faults influence.

3 UNDERSTANDING MECHANICAL EFFECT OF SEEPAGE

3.1 *The problem*

Mechanical effect of seepage it is usually important and sometimes the dominant factor in the design of civil engineering structures. Seepage flow conditions impose the system of forces resulting from them, and this is very difficult for several reasons. Firstly, subsurface flow conditions are governed by the geometry of the geological discontinuities within the rock, which is not easily defined. Secondly, they are also influenced by the flow boundary conditions, including the time factor (transient or steady flow). The third reason is that flow conditions vary with deformation of the rock mass, because fissures open or close. This point is further complicated by the fact that some deformations are produced by the seepage pressure itself. And lastly, measurements of hydraulic conductivity arc sensitive to scale effect.

Water percolating through cracks in the rock has a certain hydraulic head at all points so that the concept of the potential gradient is applicable. Seepage forces act at all points within the rock mass and are proportional to the potential gradient. However, the boundary between different geological formations or faults with breccias may constitute real impervious curtain on which the hydraulic gradient concentrate and large pressure forces act to create very unfavorable conditions.

The hydraulic conductivity of a rock mass is governed almost exclusively by the geological discontinuities whose “permeability " is much greater than that of the pores in the rock material, even though they represent a much smaller volume of voids. Therefore the structural anisotropy of the rock gives rise to anisotropic conductivity, and it also provides preferential directions along which the hydraulic forces will act. These forces ate detrimental for stability in many cases because they may reach values that are of the same order of magnitude as the other forces, such as the dead weight of the rock or the dam thrust, and they may be directed towards the free surface.

The only practical tool available to the engineer at present consists of trying to foresee every possible type of behavior and introducing it into the design analysis in the form of a model. The forces found with some schematic models can be used in the stability analysis or in the structural analysis. If this approach is neglected severe accidents may occur. In order to illustrate the significance of the rock internal forces created by seepage several case studies are presented to students. In the present paper only the first one is included, the one that refers to the technical accident at the HPP Leresti where the internal forces have displaced a large rock block thus leading to headrace lining failure.

3.2 *Failure of the Leresti pressure tunnel due to internal rock movements*

Downstream of the surge tank of the Leresti power plant the water circuit was provided by a 175 m long pressure tunnel.

sure forces leading to a large and sudden movement of a rock block. The steel lining of the pressure tunnel failed in tension and the water emerged on the slope, activating a 50 000 cum landslide which destroyed the last part of the tunnel, the valve chamber and the upper part of the penstock. A more detailed presentation is offered to students. In the depth the rock mass is divided into blocks by faults, joints and shistosity planes (Figs. 6 and 7).

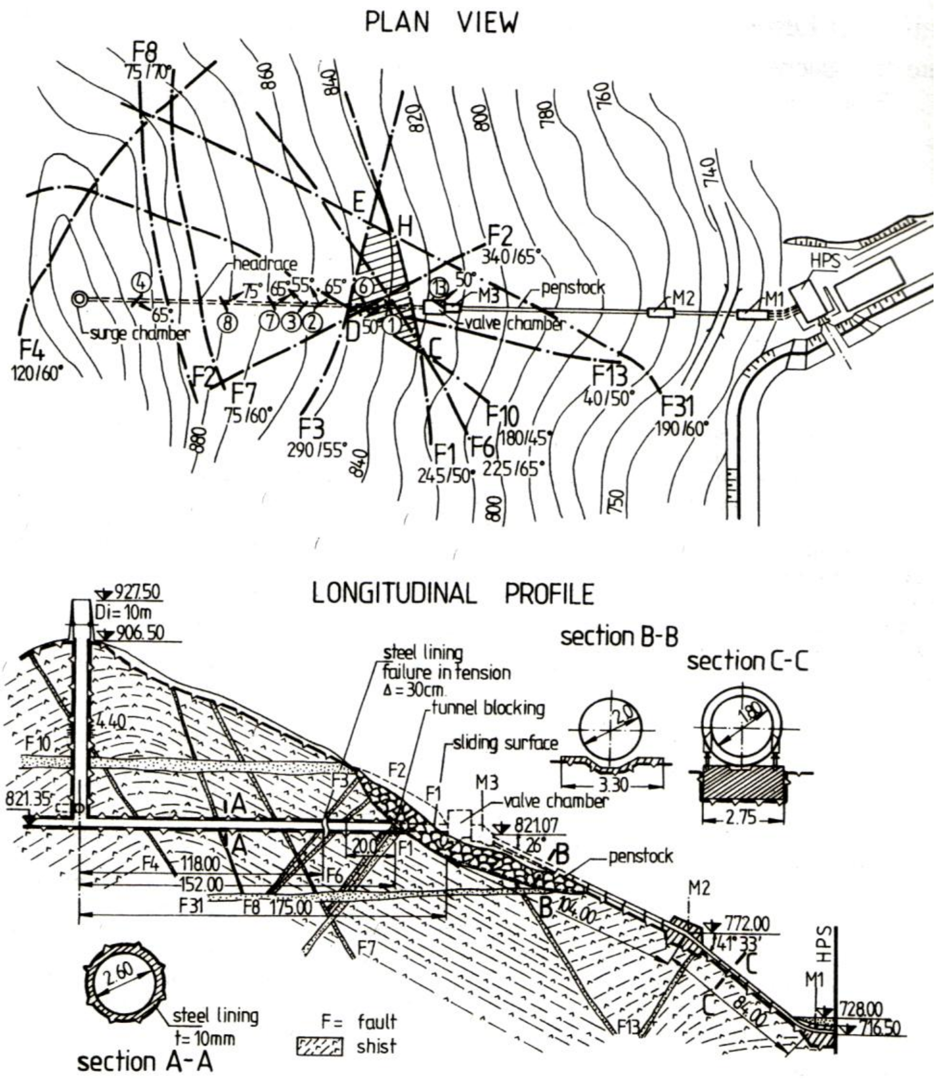


Figure 6. Hydraulic circuit of Leresti powerplant: site geology and affected works

At the exit of the tunnel a valve chamber was the house of two butterfly valves and from there a 188m-long penstock provided the connection to the powerhouse (Fig. 6). The rock mass in which the surge tank and the pressure tunnel were performed is made of quarzose-micaceous shists and is divided into rock blocks by several fault systems.

Due to faulty performance of the concrete lining extensive seepage has occurred from the shaft and the lower chambers of the surge tank, the water being stored in the rock mass. Large pressure forces were acting on the fault faces which were true impervious curtains due to their clayey breccia. The rise of the reservoir water level amplified the pres-

The rock bedding is parallel to the slope. The first set of faults (F4, F7, F8 and F13) which deep along the slope are intersected by a second one which dip inside the slope (Fl, F2, F3) and present wide zones, up to 5 m thick, of fractured rock and earth filling.

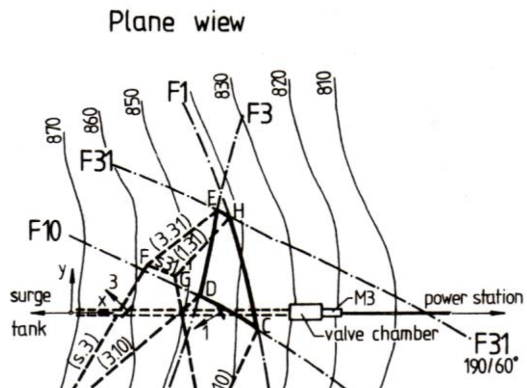


Figure 7. The instable block defined by major discontinuities

A special attention should be drawn to the following faults: fault Fl , intercepted by the pressure tunnel in the caving zone, dipping 50°inside the slope, with a 5 m-thick fault breccia; faults F2 and F3,intercepted by the pressure tunnel in the zone where the steel lining failed in tension, dipping inside the slope, with impervious fault gouges; fault F10, intercepted by the surge tank shaft, with a E-V strike, dipping 45° towards S, having a 6...7 m- fault breccia.

The rock permeability, determined by in situ tests varies largely between 4 and 33 lugeoni. The permeability is due to the fissure matrix while the fault clayey filling creates impervious screens.

The water leaked from surge tank shaft has created water pressure within the rock mass and pressure forces on the faultsF2 and F3 (Fig. 3). The 10 m rise of the reservoir water level in the last two weeks before the accident has amplified the pressure forces and a sudden movement of the rock mass downstream the fault F3 occurred. The relative movement broke the pressure tunnel lining and the water outflow along the fault faces triggering a landslide.

The displacement was initiated in the plane of fault F3 and took place along the tunnel axis, inside the rock slope. The only downstream rock discontinuity which can limit the 10 cm displaced rock mass is the fault Fl with 3 5m thick loose breccia able to allow a decimeter compression. The lateral boundaries are fault F31 on the right and fault F10 on the left. The limitation of the rock mass internal displacements was due to the fact that the initiated movement as arrested by the sudden decrease of the pressure forces when water started to flow along the face of fault F3 and by the reactive forces stabilized on the face of fault Fl.

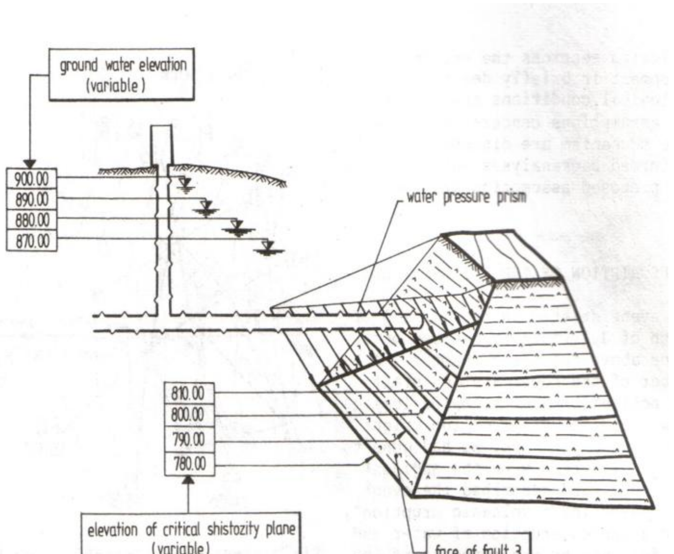


Figure 8. The computational model of the pressure forces

Several alternatives for reconstruction of the hydraulic circuit between the surge tank and the Leresti power station have been proposed. The two main alternatives are presented in Figure 9. In the first alternative the new water circuit is provided by a pressure shaft and a quasi-horizontal penstock placed in a visiting gallery. In the second alternative the above ground valve chamber and the penstock are replaced by an underground valve chamber, an inclined pressure tunnel and a short horizontal underground penstock .The second alternative is more convenient because makes better use of the unaffected upstream pressure tunnel but its feasibility requires the new underground works to be located underneath the previously unstable rock block.

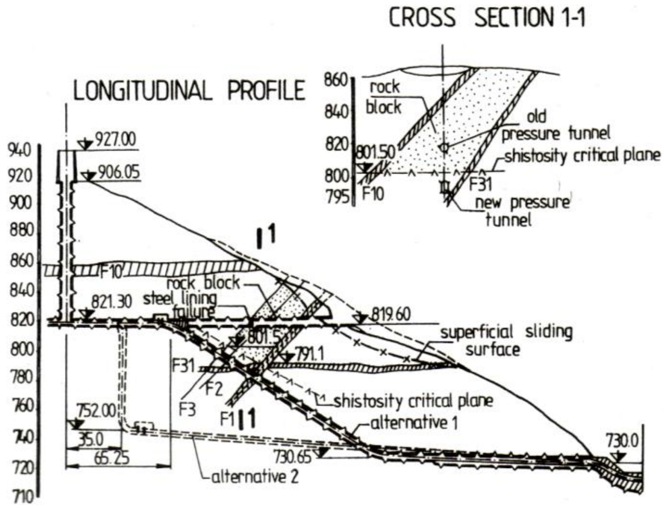


Figure 9. Alternatives for reconstruction of a new water circuit downstream of surge tank.

A parametric analysis was performed considering as variables the elevation of the shistosity - parallel critical plane, the shear strength and the ground water elevation in order to estimate the profoundness of the critical plane along which the displacements took place. The parametric study was of paramount importance for feasibility assessment of the proposed remedial works. The new water circuit downstream the surge tank which replaces the penstock with an inclined pressure tunnel was driven successfully.

4 UNDERSTANDING THE EFFECT OF EXCAVATION SEQUENCES ON THE RE-EQUILIBRIUM STATE OF THE ROCK MASS

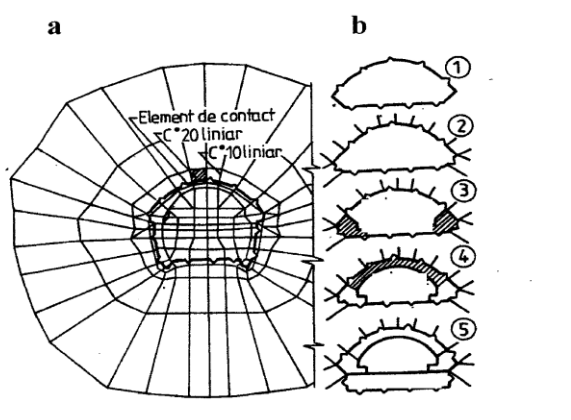
4.1 *The problem*

In accordance with the trend in the underground construction field, the excavation support is frequently performed using anchors and shotcrete. In the first stage the caverns' vaults are stabilized through such temporary support, while later on they are provided with strong reinforced concrete protection applied all along the cavern, although the temporary support was designed to withstands alone the rock pressure.

An important characteristic related to large caverns is the staged excavation. Unlike most common sized galleries or tunnels, the excavation of large caverns is not performed in full section but in stages, in order to ensure increased stability of the excavation contour up to support installation. Due to its large opening and height, the cavern vault is firstly excavated in a sequence order that provides stability during the excavation process. There is a wide array of possible sequences, but each of them is suitable for specific site situations. The vault excavation is often performed along the longitudinal axis of the cavern using alternating strip system, keeping unexcavated rock mass in between the excavated strips to provide stability. Excavation of remaining strips is performed only after the concreting of the reinforced vault corresponding to the excavated strips. In order to select the proper excavation sequence a quantitative evaluation of different alternatives is required. The stress-deformation analysis plays a double role. On one hand they evaluate the new stress state of the rock mass generated by excavation in successive stages. On the second hand, based on certain failure criteria, one has to assess whether the excavation is stable or it can be stabilized through support works. A case study presented to students aims to demonstrate the strong dependence of stress state on construction sequences.

4.2 Selection of excavation sequences for HPP Rucar cavern

The Rucar cavern is situated approximately 70 m underground, with a 16.2 m-span and a 52 m-length. The machine room is a sole cavern, whereas the units are placed in two independent shafts.



1- excavation
2 – anchoring
3 – vault base concreting
4 – vault concreting
5 - excavation

The finite elements mesh and the proposed construction sequences are presented in Figure 10. Two alternatives were analyzed. The first alternative consists of cavern open cut continuous excavation, the successive stages being excavation, anchorage, face advancement, vault support concreting, arch concreting and then the excavation of the first layer under the vault support blocks. The second alternative consists of excavation using alternating strips and rock pillars in between them and excavation of pillars subsequent to arch concreting in the excavated strip area.

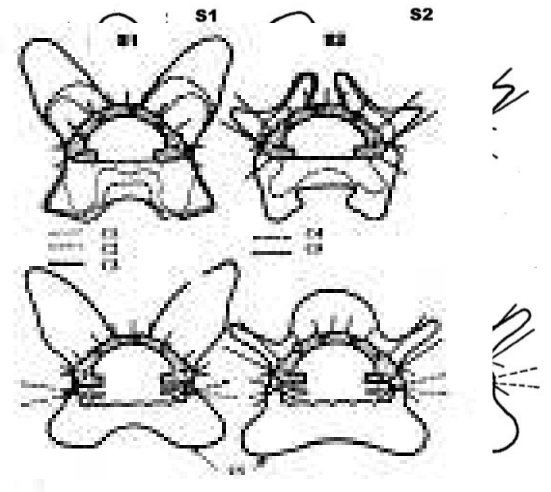


Figure 11. The results of structural analysis dedicated to selection of the performance sequences for Ruieni powerplant: S1-sequence 1; S2 – sequence 2.

The evolution of the plastic areas during construction sequences is shown in figure 11. Based on the analysis results the alternating strips and pillars alternative has been chosen, as being the only one able to ensure excavation equilibrium through rock anchoring.

5 CONCLUDING REMARKS

The paper presents only a few of the rock mechanics concepts that require a support from case studies.

In order to achieve the main scope of this approach a uniform presentation of each case is provided by using multi – media facilities.

Finally, it is important to underline that even in its modern form, rock mechanics still deals with many aspects of rock behavior that are not strictly speaking mechanical. The actual understanding of its extension needs a broad area of engineering application but this is far beyond of the student curricula.

REFERENCES

Stematiu, D. 2008. Rock mechanics for civil engineers (in Romanian). Bucharest: CONSPRESS.