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NUMERICAL SIMULATION OF ELASTIC-PLASTIC STATE OF ROCK MASS AROUND EXCAVATIONS

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Abstract: In the article one approach to determination of geo-mechanical tasks, which take into account the rock deformation breed over the strength limit, is described. Approach is adapted to solve the task with a numerical method. Verification showed that results which are got at application of this approach are adequate an analytical decision and a results of "in-situ".

Keywords: plastic flow area, equivalent stresses, rock model, floor rock heaving

1.INTRODUCTION

Simulation of the rock failure nonlinear process around the underground workings is conveniently executed, using a finite elements method (FEM). The elastic solutions are at the basis of this simulation. The algorithm considers the nonlinearity of the investigated models. Two ways are the most well known to the solution of the problem.

The first way is based on the solution of the problem of limit elastic equilibrium (Fig. 1, a). At that the components of stress and strain tensors are determined at each point of the rock mass around working.

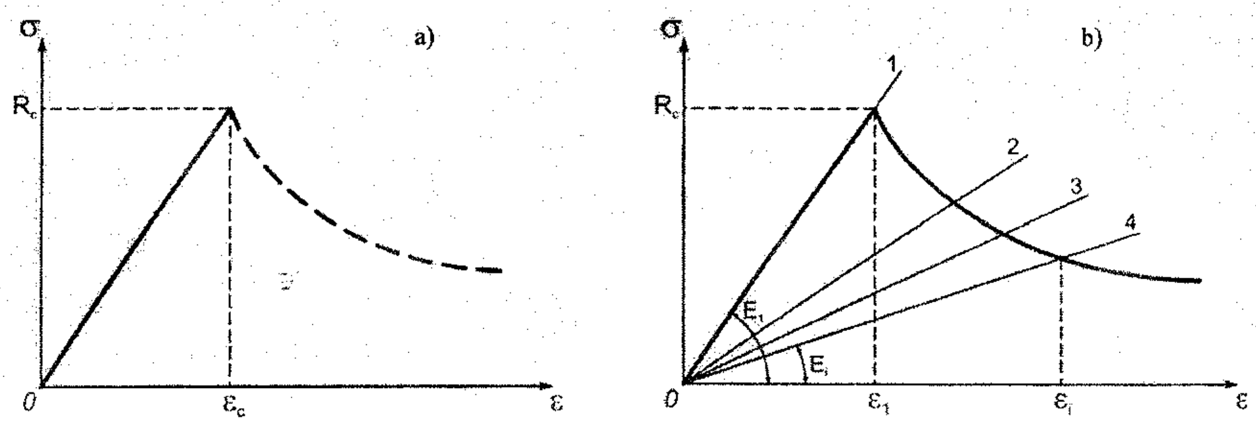

Fig 1 Models of rock medium deforming: a) limit elastic model; b) limit elastic with crossing Young's modules

Then equivalent stresses are determined and they are compared with the compressive strength. The scale effect is considered with the help structural factor. The finite elements which satisfy the condition

$$\sigma_e \geq R_c k_c \quad (1)$$

are considered as to be failed. They form a plastic flow area (PFA) around a working.

The second way assumes, that in the process of the controlled rock failure around the working on the graphs "$\sigma - \varepsilon$" the section of the decreasing curve is formed (Fig. 1, b).

The position of this section can be possibly determined with the help of the elasticity crossing modulus E_i

and be considered in the case of sequential limit elastic FEM-based solutions. The results of the solutions are summarized. This allows considering them to be the special features of the rock failure model. The dimensions of plastic flow area are just defined as in the first approach.

2. NUMERICAL SOLUTION OF ELASTIC-PLASTIC TASK

The third way for study of elastic-plastic state of rock mass around workings is offered.

It is proven that the presence of a decreasing section on the deformation diagram brings to the fact that in the plastic flow area the so-called Ȧdamar's overstability condition is not satisfied. System of equations relatively to strains becomes indeterminate and computational process stops.

There is a following assumption to account the decreasing section of the deformation curve. Any point of rock mass around the working is located in the situation of the three-dimensional stressed condition. Its level is such, that around the working the region of the rocks with the partially destroyed connections is formed. The limiting stress condition can be replaced with equivalent uniaxial condition, using any criterion of strength. The hypothesis of correspondence takes the form: the change in the equivalent stresses in the rock mass around working exactly coincides with the destruction curve of rock sample in the mode of the prescribed deformations. Let us examine the deformation diagram, consisting of three parts (Fig. 2): OA – the linear section of the elastic deformation; AB – decreasing branch of the extreme stress conditions; OC – branch of limiting deformations.

The point A corresponds to limit stresses and elasticity strains $(\sigma = R_c,\ \varepsilon = \varepsilon_c,\ \varepsilon_v = 0)$, points B and C – to stresses and strains of final disintegration ($\sigma = R_*,\ \varepsilon = \varepsilon_*,\ \varepsilon_v = \varepsilon_{v(*)}$).

Assume that with the elastic deformation in the section OA on a certain step m the limiting value of the stress R_c is exceeded so that the point $A_m(\varepsilon_m, \sigma_m)$ is the end point of step. In accordance with function, describing the decreasing branch of the stress-strain diagram, there is the point $B_m(\varepsilon_m, R_m)$ in this decreasing branch AB that can be determined. Then the values ε_m, $\varepsilon_{e(m)} = R_m/E$, $\varepsilon_{d(m)} = \varepsilon_m - \varepsilon_{e(m)}$ characterize complete elastic and dissipative strains at the point B_m.

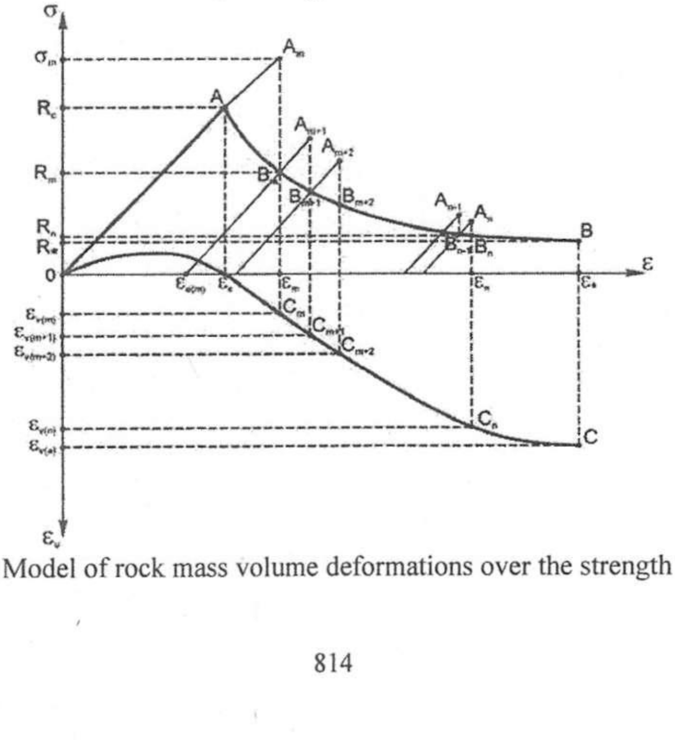

Fig 2 Model of rock mass volume deformations over the strength limit

Confine strain at the point B_m is defined by using the curve of confine strain ε_v. The point c_m which lies on the confine deformations curve and value $\varepsilon_{v(m)}$ corresponds to the point B_m.

At the step $m+1$ is a newly produced elastic solution, but this time on the basis of the point B_m. Solution process iterates, until the final step n with the finite strains ε_n at the point $B_n(\varepsilon_n, R_n)$ will not be attained. Thus, during the entire solution process the point of emergence into the decreasing sections of the diagram occurs.

To solve the boundary problem, the approach presented above is generalized in the case of the triaxial stress condition. Detailer description of numerical decision algorithm can be found in the articles (Shashenko, 2001, 2005).

The analytical studies are the basis of the algorithm of the numerical solution of different elastic-plastic tasks.

3. CHECKING ADEQUACY OF NUMERICAL ALGORITHM

The adequacy of the numerical algorithm was checked for correspondence to the verification analytical solution given in (Shashenko, 2001). Toward this end the results of numerical solutions were compared with the natural measurements of the dimensions of the plastic flow area and displacements of the working contour.

Fig. 3 shows the dependences of the radius of plastic flow area R_L in reference to the radius of a working R_0 by the index of excavation conditions $\frac{R_c k_c}{\gamma H}$, constructed according to the real measurements (pos. 1-3), and also by the results of numerical calculations for the limit elastic model (pos. 5), the model with the variable elasticity modulus (pos. 6) and model, which considers loosening of rock mass beyond the limit of strength (pos. 7). As it is seen from the figure, in this case all models give the results, sufficiently close to the curve, which approximates the results of multiple natural measurements (pos. 4) and to the curve, built according to the results of analytical calculations (pos. 8).

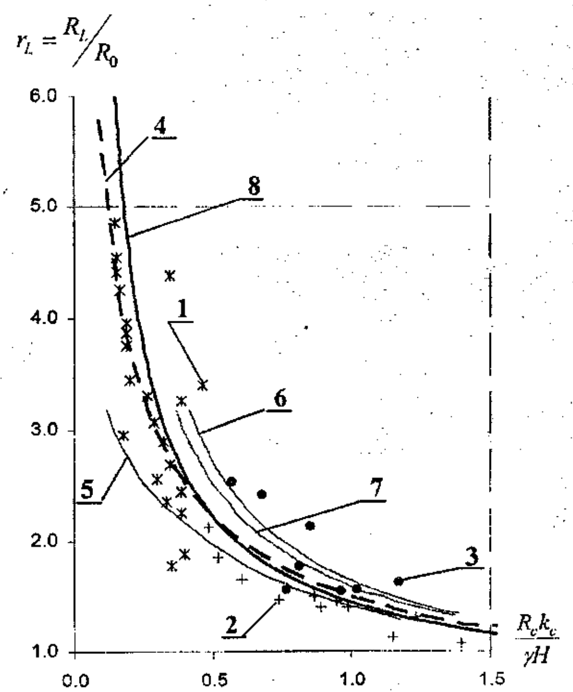

Fig 3 Comparison of results of real measurement of PFA relative radius, and results of numeral modeling for different models of rock deforming

Fig. 4 shows dependences of relative displacements on the working outline for the same index of excavation conditions. Differences in the results, obtained for different rock models, are obvious: the least adequate is the limit elastic model (pos. 3), the most adequate are the results of calculations for the model with the decreasing curve (pos. 1). Intermediate position is occupied with the model of environment with the variable elasticity modulus (pos. 2). The curve of pos. 4 is the result of analytical calculations.

The third elastic-plastic model of the rock failure around underground workings was used for solving different geomechanical tasks. One of them is the task of modeling of a working process rock floor heaving. The diagram for the task analysis is shown in Fig. 5.

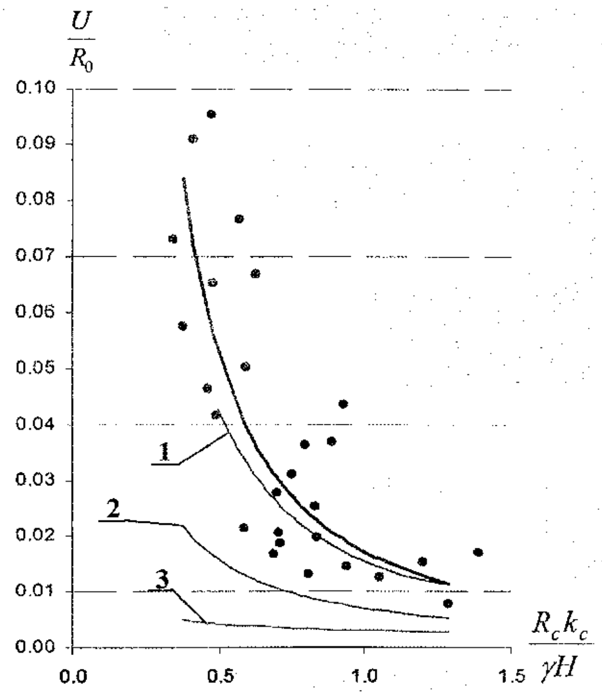

Fig 4 Comparison of results of the real measurement of contour working relative displacements, and results of numeral modeling for different models of rock deforming

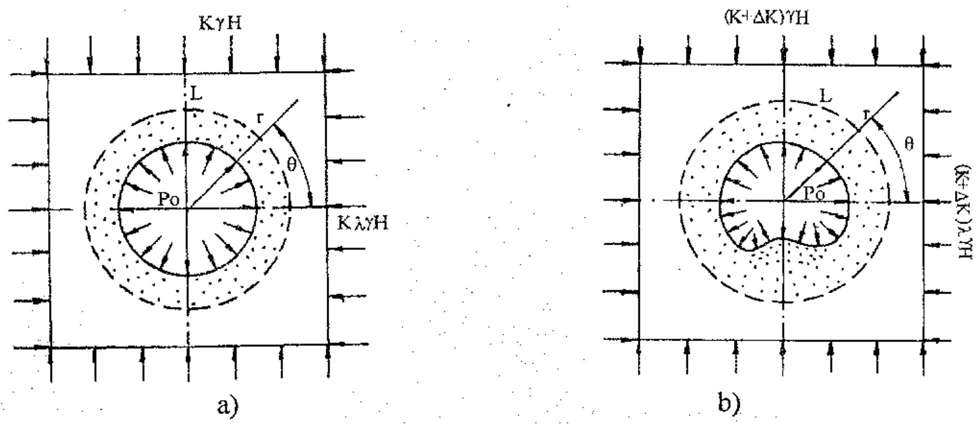

Fig 5 Diagram for analysis of elastic-plastic stability losses: a) initial state of system; b) the disturbing state of system

The carried out research with use of physical models showed that stability loss for elastic-plastic equilibrium rock around underground workings is less (Fig. 6).

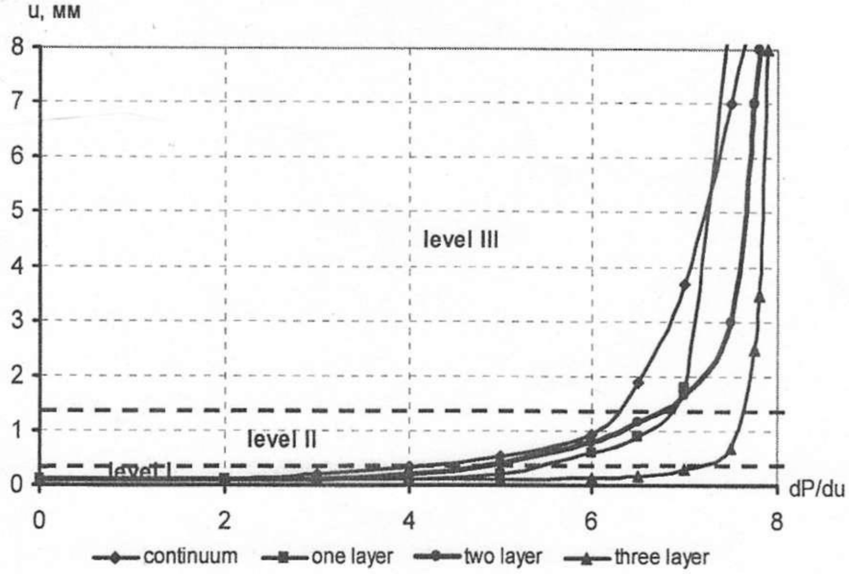

Fig 6 Dependence of floor heaving on intensity of exterior loading

4. NUMERICAL MODELING OF DEFORMATION PROCESS

Investigations "in-situ" show that geomechanical processes around excavations develop in such a way. There are three different levels of a stress strain state of rock mass.

The first is a level of elastic deformations. At the second level the area of plastic deformations (APD) is formed.

At reaching critical size of this area the third level occurs. There is fast transition of the system from the second energy level to the third one. The transition is accompanied by major displacement of excavation contour. This process is termed as lossof elastic-plastic stability, or rock floor heaving.

The numerical modeling of a rock floor heaving process is carried out with use of finite element method (FEM) at the basis of the third model of the rock failure.

Calculations start with solution of the problem for a homogeneous rock mass. Inhomogeneity of a rock mass structure increases in the course of solving. The generalized scheme is presented in Fig. 7.

It is shown, that size of APD in floor exceeds original one by 2-3 times at value of a floor heaving equal to 0,3 m (Fig. 8).

Comparison of actual displacement in a dukeway № 2 floor in mine "Belozerskaja" with value of floor displacement which has been received at numerical simulations, have shown their good correspondence. Discordance of calculations and on-site results was 10-15 %.

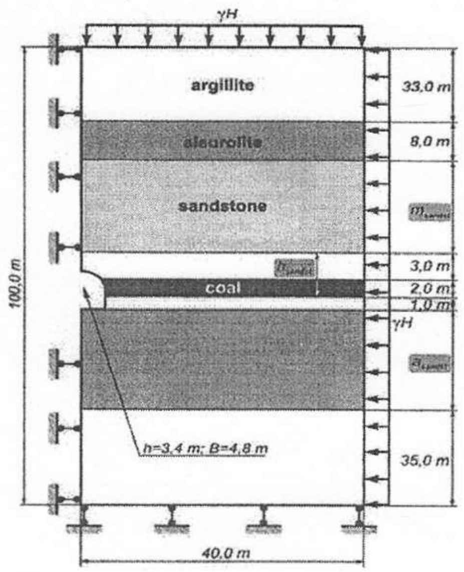

Fig. 7. The generalized solution scheme

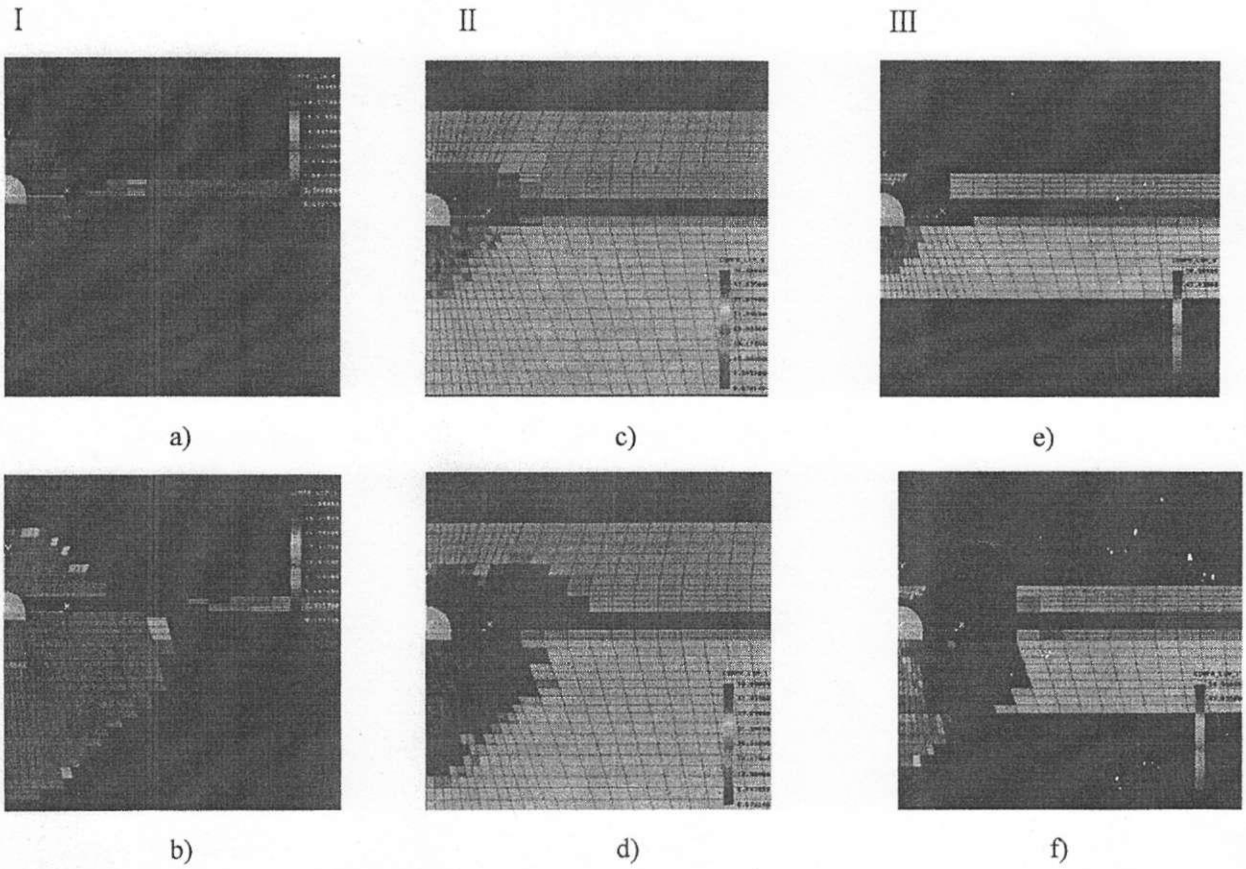

Fig 8 Configuration of APD: a, c, e - up to the floor heaving; b, d, f – after the floor heaving, I – homogeneous rock mass including a coal seam; II - rock mass including a coal seam and seam of sandstone in roof (distance between coal seam and sandstone is $h_{sandstt}$ = 10,0 m); III - rock mass including a coal seam and seam of sandstone is floor (distance between coal seam and sandstone is a_{sandst} = 7,0 m)

5. CONCLUSION

Being based on the described investigations, the following conclusions have been reached:

1. Obtained results of research do not contradict the existing physical concept about rock deformation around a working.
2. The comparison of the results of "in-situ", analytical and numerical studies proves the adequacy of the rock mass deformation model which is based on the analogy with deformation of a rock sample in the mode of controlled failure.
3. This rock deformation model can be used for investigation the state of "working- rock mass" geomechanical systems and obtaining the correct results.

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