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Analysis of common geological models of materials

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Abstract. The paper analyzes the physical models used in the study of the stressed state of geological rocks. Common physics models include: 1) soil and foam; 2) pseudo-tensor; 3) geological; 4) Schwer-Murray; 5) continuous surface of the cap; 6) Mohr-Coulomb; 6) connected stone. A graphical representation of the description of the soil and foam model is given. At the initial stages of loading with small deformations, the model behaves linearly elastically. When the level of deformations increases, it turns into a non-linear model. The graphical presentation of the pseudo-tensor model reflects two modes of operation of the model depending on the physical properties of the material. The geological model is one of the subspecies of the geological cap model and is used in solving geo-mechanical problems, as well as in modeling such materials as concrete. The graphic representation of the geological model is described by three curves, the functions of which are given in the paper. The Schwer-Murray model is an extended version of the geological model that includes visco-plasticity to calculate velocity effects and damage mechanics. The prize is designed for the study of such materials as soils, concrete and rocks. An improved Schwer-Murray model is the continuous cap surface model (CSCM), the yield surface of which is defined by three stress invariants. The Coulomb-Mohr model is intended for the study of solid elements, thick shells and SPH particles. It is used to represent cohesive or non-cohesive rocks, soils, clastic cemented rocks, sandy soils, and other granular materials. The joint stone model is analogous to the Drucker-Prager and Coulomb-Mohr models. The oriented crack model is used to model brittle materials (ceramics) or porous materials, such as concrete, which undergo failure due to high tensile loads. Basically, it can be an isotropic elastoplastic or elastic material with an oriented crack.

Keywords: pseudo-tensor, invariant, yield strength, plastic deformation, destruction, strengthening, concrete, rocks, soil.

INTRODUCTION

Today, there are a number of classic rheological models that are used in the study of the model of the machine-working environment system. Such models include the following: Hook, Saint-Venant, Newton, Prandtl, Maxwell, Voigt, Kelvin, Bingham, Shvedov. For example, a perfectly elastic body can be described using the Hooke model. This model can be used in calculations of springs, vibration limiters, etc. [1].

The Saint-Venant model describes a perfectly plastic material or represents the dry friction of a rigidly plastic material. To some extent, such a model was used to describe the concrete mixture [2].

Newton's model describes a perfectly viscous body and characterizes the operation of frictional forces within the body or between separate bodies under the condition that the stresses are proportional to the gradient of the speed of the applied load.

In general, it should be noted that real bodies may exhibit different physical properties under loads. In some cases, these sets of physical properties can manifest simultaneously. That is, bodies with one physical property are rarely found in nature.

The process of studying the machine-environment model is carried out through mathematical or physical modeling. Classical mathematical modeling consists in displaying processes due to analytical dependencies. However, today, with the development of computerized and robotic systems, machine models have become more complex due to the combination of different process control systems. On the other hand, efforts to increase energy efficiency stimulate research and implementation of synergistic methods of interaction of working bodies of machines with the working environment. In addition, there are ongoing studies related to the attempt to more accurately describe the model of the working environment. All this contributes to greater complexity of physical models of the working environment.

A significant breakthrough in the study of solid medium mechanics was the introduction of numerical modeling methods. Of course, at the initial stages, such methods were cumbersome in terms of the use of human resources. However, with the rapid development of computer technology, these numerical methods have become widespread. The most common numerical method for calculating working environments today is the finite element method (FEM). However, this method has a number of disadvantages, especially when studying discrete environments. Disadvantages of the finite element method are partially eliminated by the discrete element method (DEM).

The rapid development of numerical methods is facilitated by their implementation in modern computing systems. Today, there is a large number of software complexes that allow you to perform finite element calculations with sufficient accuracy. The most common platforms include: Ansys, Nastran, Abaqus, etc. If we consider the problems of formation and propagation of cracks, there are separate software solutions that can perform relatively detailed calculations of models of environments with the presence of cracks in them. These software products include: Warp3D, Franc3D, Franc2D, Afgrow, Flac3D, FsaCrack. EDEM is a common software package for modeling by the method of discrete elements.

Thus, we see a wide range of software products, which is not completely exhaustive. It should be noted here that these software products are successfully used for modeling work environments. Based on their long-term use, libraries of physical models of materials have been compiled.

In this work, we will consider the common physical models used in the modeling of rocks. Such models include [2]: 1) soil and foam; 2) pseudo-tensor; 3) geological; 4) Schwer-Murray; 5) continuous surface of the cap; 6) Coulona-Mora; 6) connected stone.

At the same time, the following types of models are widely used in the modeling of geological rocks: 1) brittle fracture; 2) Johnson-Holmquist for concrete; 3) primed concrete; 4) hysteresis soil; 5) Ramberg-Osgood; 6) modified Drucker-Prager model; 7) destruction of soil and foam; 8) oriented crack; 9) destruction of concrete; 10) Winfrit for concrete; 11) reinforced concrete shear wall; 13) concrete beam.

GOAL AND PROBLEM STATEMENT

Conduct an analysis of common physical models of the working environment for further description and determination of rational parameter values when interacting with the working bodies of crushing machines.

MAIN PART

Soil and Foam Model. It is a relatively simple model for describing soils, concrete or deformable foam [4]. This model was created for modeling elastic-plastic materials. The flow function is described by the following equation:

$$\psi = I_2 - (a_0 + a_1 p + a_2 p^2), \qquad (1)$$

where p – pressure; a_0, a_1, a_2 – constants; I_2 – the second invariant of the deviatoric stress tensor.

The behavior of the model is as follows - at the initial stages of loading, with small deformations, the model behaves linearly elastically. When the level of deformations increases, it turns into a non-linear model. In equation (1), the second invariant is determined based on the following dependence:

$$I_2 = \frac{1}{2} S_{ij} S_{ij}, \qquad (2)$$

where S_{ij} – deviatoric stress tensor.

The deviatoric stress tensor is determined according to the following relationship [4]:

$$S_{ij} = \sigma_{ij} + (p + q)\delta_{ij}, \qquad (3)$$

where σ_{ij} - stress tensor; δ_{ij} - the Kronecker coefficient, which takes the value 1 under the condition that the indices are equal to each other and zero for different values of the indices; q – bulk viscosity, as viscosity is not included in this model, then q=0.

It should be noted here that dependencies (2) and (3) are generalized to describe the volumetric loaded state of the material. So, for example, when describing soils, one should take into account stresses and deformations that occur along different axes, that is, a stressed state that will repeat the conditions of soil deposition. A simple model of material destruction in the crushing chamber takes into account uniaxial loading.

Under the conditions of a triaxial stress state, the fracture stress will be equal to:

$$\sigma_p = \sigma_x - (\sigma_z + \sigma_y). \qquad (4)$$

For simplicity, we assume that the stresses arising from the boundary material along the z and y axes are equal. Then the average failure stress will be equal to:

$$\sigma_c = (\sigma_x + 2\sigma_z)/3. \qquad (5)$$

The second invariant of the deviatoric stress tensor, taking into account the condition of equality of stresses along z and y, can be written as follows:

$$I_2 = \frac{1}{2}\left[\left(\frac{2\sigma_{x-z}}{3}\right)^2 + \left(\frac{\sigma_{z-x}}{3}\right)^2 + \left(\frac{\sigma_{z-x}}{3}\right)^2\right], \qquad (6)$$

To determine the unknown coefficients a_0, a_1, a_2 in (1) a graph of the dependence of the values of the second invariant of the deviatoric tensor I_2 on the average stress is constructed σ_c. Further, on the basis of quadratic regression, the unknown coefficients of equation (1) are determined.

Under the condition of uniaxial loading, the second invariant of the deviatoric tensor can be determined from the following relationship - $I_2 = \frac{1}{3}\sigma_y^2$.

Then dependence (1) can be rewritten as follows [3]:

$$\frac{1}{3}\sigma_y^2 = (a_0 + a_1 p + a_2 p^2). \qquad (7)$$

The dependence of pressure on the change in material volume is shown in Fig. 1.

In general, it can be noted that this model is ideal for describing the loaded state of soils, and can also be used for many rocks.

Pseudo Tensor Model. A material model that was created for the analysis of embedded steel-reinforced concrete structures subjected to impulse loads. This model has two modes depending on the physical properties of the material. The first mode is a normal flow curve, the second mode includes two flow curves that are functionally dependent on pressure, and also includes switching parameters between the curves. For both modes, it is possible to set the LCP load curve, which is the deformation rate parameter for the plasticity curve.

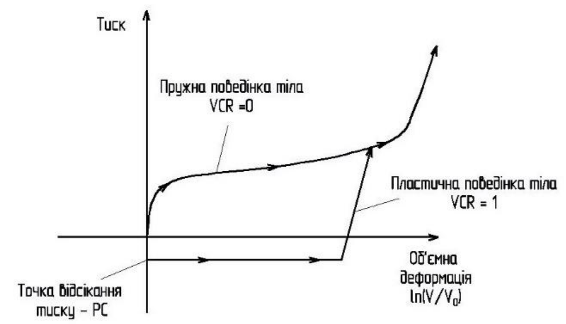

Fig.1. Dependence of pressure on volume change for the material for the soil and foam model

We will separately consider the modes of the pseudo-tensor model. The first mode is well suited for describing standard geological models, such as the Mohr-Coulomb yield surface with the Treska boundary, Fig. 2.

The Mohr-Coulomb model with the Treska limit in combination with the equation of state was well-recommended for describing the interaction of soil with a reinforced concrete structure [3]. In order to set mode 1 of the model, it is necessary to accept the pseudo-tensor, that $a_0 = a_1 =$

$a_2 = b_1 = a_{0f} = a_{1f} = 0$, then it is necessary to specify the corresponding pressure values and the corresponding values of the yield point [3]. The parameters related to the properties of the reinforcement, the initial yield strength and the tangential modulus must also be set to zero.

The second mode. A combination of two models - destruction and damage. This mode uses two pressure-dependent flow curves in the following form:

$$\sigma_y = a_0 + \frac{p}{a_1 + a_2 p}. \quad (8)$$

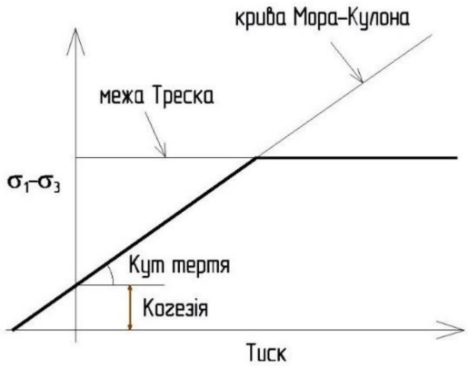

Fig. 2. Mohr-Coulomb curve with Tresca limit

The first yield curve best describes the maximum yield point, while the second curve better describes the destruction of the material. Here there is a considerable number of variations in setting the appropriate curve for the material. Let's consider the options given in the source [40].

Option 1. Simple destruction due to stretching. It is necessary to set coefficients a_0, a_1, a_2, a_{0f}, a_{1f} . Coefficient b_1 takes the value zero. In this case, the yield point of the aluminum material is taken as the maximum value on the yield curve, until the maximum principal stress in the material (σ_1) does not exceed the ultimate tensile stress ($\sigma_в$). When the moment comes $\sigma_1 > \sigma_в$ the yield strength is reduced by some delta, which is equal to the distance between the curves (the curve of the maximum yield strength and the failure curve) in each of 20 clock steps. That is, after 20 clock steps, the yield point will be determined by the material's destruction curve.

Option 2. Tensile failure plus scaling of plastic deformation. The difference from option 1 lies in the introduction of the scale factor η as a function of the effective plastic deformation. The program calculates the coefficient η, and then determines the yield strength based on the following relationship [3]:

$$\sigma_п = \sigma_в + \eta\left(\sigma_{max} - \sigma_в\right), \quad (9)$$

where σ_{max} and $\sigma_в$ determined based on dependency (8).

This version of the model well describes a material that is strain-hardened or strain-softened, such as concrete.

Option 3. Tensile failure with damage scaling. The change in yield strength as a function of plastic deformation occurs due to physical mechanisms such as internal cracking. The degree of cracking is affected by hydrostatic pressure. This mechanism can lead to a "restriction" effect in the behavior of concrete. To take this phenomenon into account, the "damage" function is introduced, which has the following form:

$$\lambda = \int_0^{\varepsilon p} \left(1 + \frac{p}{\sigma_в}\right)^{-b_1} d\varepsilon^p. \quad (10)$$

Coefficients are set to define the model a_0, a_1, a_2, a_{0f}, a_{1f} та b_1 and parameter η, as a function λ.

Geological Cap Model. The model is used in solving geomechanical problems, as well as in modeling materials such as concrete. The model is based on the theory of the geological cap, to which the theory describing nonlinear kinematic strengthening [42], [43], [44] is implemented. In fig. 3 graphical interpretation of the geological model is displayed.

Mathematical formulation of the model presented in fig.3 is given in terms of invariants of the stress tensor. The square root of the second invariant of the stress deviator tensor is determined as follows:

$$\sqrt{J_{2D}} = \sqrt{\frac{1}{2} S_{ij} S_{ij}}. \quad (11)$$

On the graph fig. 3 the root of the second invariant of the stress deviator tensor is a measure of distortion or shear stress. While the first stress invariant J_1 displays the stress tensor.

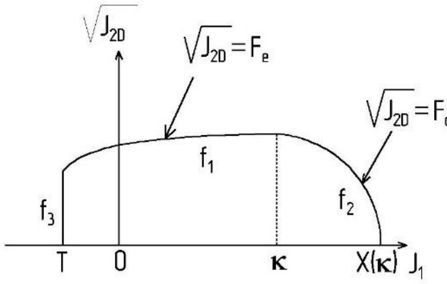

Fig. 3. The curve describing the geological cap model

The geological cap model in fig. 3 is described by three different surfaces.

The first surface (f_1) is a curve of surface destruction. The function of the first curve is written as follows:

$$f_1 = \sqrt{J_{2D}} - \min[F_e(J_1), T_{mises}], \qquad (11)$$

where $F_e(J_1) = \alpha - \gamma \exp(-\beta_1 J_1) + \theta J_1$;

$T_{mises} = \left| X(\kappa_n) - L(\kappa_n) \right|$.

The surface f_1 is a fixed space that does not strengthen if it does not have kinematic strengthening.

Surface f_2 represents a geological cap and is functionally written as follows:

$$f_2 = \sqrt{J_{2D}} - F_c(J_1, K), \qquad (12)$$

where F_c is determined based on the following dependence

$$F_c(J_1,\kappa) = \frac{1}{R}\sqrt{\left[X[\kappa] - L(\kappa)\right]^2 - \left[J_1 - L(\kappa)\right]^2}, \qquad (13)$$

where is the point of intersection $X(\kappa)$ with axis J_1 is determined based on the following dependence:

$$X(\kappa) = \kappa + RF_e(k), \qquad (14)$$

In turn, $L(\kappa)$ it is determined on the basis of a system of equations:

$$L(\kappa) = \begin{cases} k \text{ if } \kappa > 0; \\ 0 \text{ if } \kappa \le 0. \end{cases} \qquad (15)$$

The hardening parameter κ is related to the plastic volume change by the following relationship:

$$\varepsilon_v^p = W\left\{1 - \exp[-D\left(X(\kappa) - X_0\right)]\right\}, \qquad (16)$$

The third surface is the stress cut surface, which is determined by the following function:

$$f_3 = T - J_1, \qquad (17)$$

where T – input parameter of the material that determines its maximum hydrostatic tension.

Condition of plastic behavior of the body:

$$\dot{\lambda}_i f_i = 0;\ i = 1,2,3;\ \dot{\lambda}_i \ge 0. \qquad (18)$$

where λ_i – plastic consistency parameter for the i-th surface. If f_i <0 then, $\dot{\lambda}_i$ = 0 and the response will be elastic. If f_i >0 then is active i-th surface and parameter λ_i is based on dependency $\dot{f}_i = 0$ [4].

The advantages of the geological model are the ability to control the amount of expansion that occurs under the action of shear loading. For example, in the Coulomb-Mohr and Drucker-Prager models, plastic expansion continues as long as shear loads are applied to the model, as a result of which, in many cases, much greater expansion is created than in experimental models. The law of hardening in the geological model allows the surface f_2 to compress until it crosses the failure zone at the point $X(\kappa)$, after which the movement stops.

Control parameters that affect the shape of the curve f_2 allows to better reproduce the physics of the process.

Another advantage of the geological model is the ability to simulate plastic compaction. In such models as Coulomb-Mohr and Drucker-Prager, the volumetric response to deformation is purely elastic. While in the geologic model, the volumetric response is elastic in the range up to the point κ. In turn, plastic volumetric deformation (compaction) is generated at a rate controlled by the law of hardening. The inclusion of kinematic strengthening leads to hysteresis dissipation of energy under conditions of cyclic loading. The work [5] considers nonlinear kinematic strengthening for the f_2 surface under the conditions that the C and N parameters are not zero. Moreover, the f_2 curve was replaced by a number of yield curves. The variable C is the kinematic strengthening coefficient, and N is the kinematic strengthening parameter.

A geological model contains a number of parameters that must be selected to represent a particular material and which are based on experimental data. Parameters α, β, θ and γ are usually estimated by fitting a curve to fracture data taken from triaxial compression tests. Parameters W, D and X_0 are determined from the law of surface hardening f_2. Parameter W represents the porous fraction of the uncompressed sample, D determines the slope of the initial load curve during hydrostatic compression. The value of X_0 is the ratio of the major axis to the minor quarter of the ellipse defining the surface f_2. In [6] additional recommendations are given for the selection of geological model parameters based on experimental data.

Schwer-Murray model. This model is an extended version of the geological model that includes viscoplasticity to calculate velocity effects and damage mechanics. The main provisions of the model are outlined in the source [7]. The Schwer-Murray model is designed for the study of materials such as soils, concrete and rocks.

The main parameters of the model are: density, shear modulus; volumetric module; Grüneisen coefficient (usually taken as zero); impact speed parameter; shear failure parameters (α,β,γ,θ); damage mechanics parameter; parameters of kinematic hardening deformation (N^{α},c^{α}); the initial ellipticity of the cap surface; the principal stress tensor J_1; parameter of strengthening of compaction due to shear stresses; plastic damage mechanics parameters; parameters of plastic volumetric deformation (W, D_1,D_2); the maximum permissible increase in deformation; parameters of brittle fracture mechanics; scalable torsion; scaling for three-axis expansion; viscoplastic relaxation time.

The parameter should be specified separately FAILFL, which determines whether damage accumulation should be applied to the overall stress tensor σ_{ij} or to the deviatoric stress tensor S_{ij}. In addition to this parameter FAILFL used in the program LS-DYNA to remove completely damaged material destruction elements.

As noted in the source [3] a more advanced Schwer-Murray model and at the same time its extension is a model called CSCM (Continuous Surface Cap Model).

Model CSCM is a model of a smooth continuous surface of the cap. It is used for modeling solid materials. In the program LS-DYNA for this model, it is possible to set your own material parameters [3]. Graphic representation of the model CSCM shown in fig. 4.

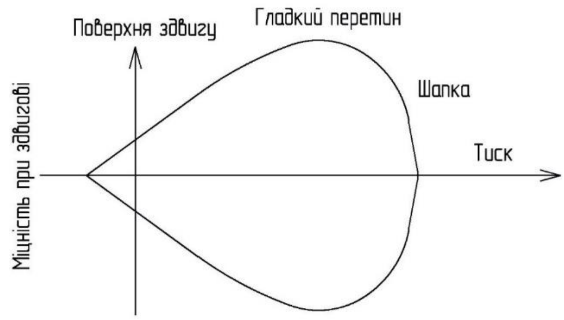

Fig. 4. Model CSCM

As can be seen from fig. 4 model CSCM has a smooth transition between the yield surface and the shear hardening cap. Velocity effects are modeled using viscoplasticity. A complete description of the model is given in the source [8]. The yield surface is defined by three stress invariants:

$$J_1 = 3P;\ J_2 = \frac{1}{2}S_{ij}S_{ij};\ J_3 = \frac{1}{3}S_{ij}S_{ik}S_{ki}. \quad (19)$$

where J_1 – the first invariant of the stress tensor; J_2 – the second invariant of the deviatoric stress tensor; J_3 – the third invariant of the deviatoric stress tensor; P – pressure; S_{ij} – deviatoric stress tensor.

The yield function is related to the three invariants (19) and the cap strengthening coefficient κ as follows:

$$f\left(J_1, J_2, J_3, \kappa\right) = J_2 - \zeta^2 F_f^2 F_c. \quad (20)$$

where F_f^2 – failure surface from shear loads; F_c – cap strengthening surface; ζ – Rubin's function.

The strength of concrete is modeled by the shear surface in tension and low all-round pressure regimes:

$$F_f\left(J_1\right) = \alpha - \lambda \exp\left(-\beta J_1\right) + \theta J_1. \quad (21)$$

where are the parameters α, β, λ, θ are determined by adjusting the surface of the model to experimental measurements of triaxial compressive strength carried out on simple concrete cylinders.

Ruby's scale function ζ determines the strength of concrete for any stress state relative to the strength for triaxial compression, in the form ζF_f. That is, the strength in triaxial torsion is determined as Q_1F_f. In turn, strength in triaxial tension – Q_2F_f. In turn Q is defined as:

$$Q_1 = \alpha_1 - \lambda_1 \exp(-\beta_1 J_1) + \theta_1 J_1; \quad Q_2 = \alpha_2 - \lambda_2 \exp(-\beta_2 J_1) + \theta_2 J_1. \tag{22}$$

Concrete strength is modeled using a cap surface and a shear surface in low to high pressure regimes. The cap is used to model the plastic volume change. The isotropic hardening cap is a function that consists of two parts and can take the value 1 or the shape of an ellipse:

$$F_c(J_1,\kappa) = 1 - \frac{[J_1 - L(\kappa)]\left[|J_1 - L(\kappa)| + J_1 - L(\kappa)\right]}{2[X(\kappa) - L(\kappa)]^2}. \tag{23}$$

Cross section of the cap with the axis J_1 is at a point X(κ) and depends on the ellipticity factor R:

$$X(\kappa) = L(\kappa) + RF_f[L(\kappa)], \tag{24}$$

Volumetric plastic deformation is determined by a similar dependence to (16) in which the parameters appear D_1 та D_2, which determine the shape of the volume strain pressure curves.

Under unconfined compression, the stress-strain state of concrete exhibits nonlinearity and expansion before the peak. This behavior is modeled by the initial shear yield surface N_HF_f, which hardens until it coincides with the final shear yield surface F_f. Parameter N_H determines the start of strengthening. The second parameter C_H determines the rate of hardening.

The model includes a scalar fracture parameter d, which transforms the viscoplastic stress tensor without fracture into the stress tensor with fracture. Damage accumulation is based on two distinct theories, namely brittle and plastic failure. Plastic failure is determined on the basis of common components of deformation:

$$\tau_c = \sqrt{\frac{1}{2}\sigma_{ij}\varepsilon_{ij}}, \tag{25}$$

Stress components σ_{ij} in the equation (25) are elastoplastic and are designed to apply damage and speed effects

Brittle fracture depends on the main deformation as follows:

$$\tau_t = \sqrt{E\varepsilon_{max}^2}. \tag{26}$$

As damage accumulates, the parameter d increases from an initial value of zero to one based on the following dependencies:

Plastic failure:

$$d(\tau_t) = \frac{0.999}{D}\left[\frac{1+D}{1+De^{-C(\tau_t-\tau_{0t})}} - 1\right], \tag{27}$$

Brittle failure:

$$d(\tau_t) = \frac{d_{max}}{B}\left[\frac{1+B}{1+Be^{-A(\tau_t-\tau_{0t})}} - 1\right], \tag{28}$$

where are the parameters A, B, C та D specify the shape of the softening curve, which is constructed in stress-displacement or stress-strain coordinates. Parameter d_{max} represents the maximum level of damage that can be achieved.

Coulomb-Mohr model. Designed for the study of solid elements, thick shells and SPH particles. Used to represent cohesive or non-cohesive rocks, soils, clastic cemented rocks, sandy soils, and other granular materials.

A simple soil model is simulated by defining five parameters: density, shear modulus, Poisson's ratio, friction angle, cohesion value (shear strength at zero normal stress), dilation angle (radians).

The yield surface is given by the equation:

$$\tau_{max} = C + \sigma_n \tan(\varphi), \tag{29}$$

where C – parameter that determines of the cohesion; φ - friction angle; σ_n – normal tension; τ_n – maximum shear stress.

The plastic potential function has the form:

$$\beta\sigma_{max} - \sigma_{min} + \text{const}, \tag{30}$$

where β is determined based on the following dependence

$$\beta = \frac{1+\sin(\psi)}{1-\sin(\psi)}. \quad (31)$$

Plastic deformation is determined:

$$\varepsilon_{п} = \sqrt{\frac{2}{3}\varepsilon_{pij}\varepsilon_{pij}}, \quad (32)$$

The angles of friction and expansion φ and ψ can change depending on the plastic deformation (corresponding parameters LCPHIEP and LCPSIEP). For modeling strongly consolidated materials with large shear strains, as the strain increases, the expansion angle usually decreases to zero, and the friction angle decreases to a lower value. Also, the shear modulus can decrease under similar conditions. To define parameters LCPHIEP and LCPSIEP tabular values must be loaded.

Load curves LCCPDR, LCCPT, LCCJDR and LCCJT allow additional cohesion to be added as a function of time. Additional cohesion is intended for use in the initial stages of analysis to ensure that gravity or other loads are applied without cracking or yielding, and to control the cracking or yielding process. If the parameters LCCPDR, LCCPT, LCCJDR and LCCJT are not specified, then additional cohesion is not applied [3].

Parameter LCSFAC allows you to set the strength factor as a function of time. This function is designed to gradually reduce the strength of the material in order to study the safety factors. To set the coefficient values, it is necessary to load the load curve. If there is no curve, it is possible to set a constant coefficient of 1. When setting values greater than one, problems with stability may arise. An alternative solution for introducing time-dependent properties is the ability to define time functions in the parameters GMOD, CVAL and PHI using load curves for parameters LCGMT, LCCVT and LCPHT in accordance [3].

The parameter is responsible for choosing a soil or rock model NJOINT. Parameter for soil NJOINT is taken as equal 0. Modeling of rocks is carried out according to a similar mechanism that is embedded in the model - connected stone.

When modeling rocks, the parameter LOCAL must be equal 0. This parameter corresponds to the coordinate system in which the angles will be determined – DIP та DIPANG.

In the model, it is possible to set the anisotropy due to the parameter ANISO, which applies to elastic shear stiffness in global planes XZ and YZ.

Model of Jointed Rock. This material model is similar to the Drucker-Prager and Coulomb-Mohr models. In order not to take into account the flow surfaces, it is necessary to set the value of the parameter ELASTIC equal 1.

Form factor of the material RKF it is not desirable to set it to a value below 0.75.

Just as in the Coulomb-Mohr model, it is possible with the help of load curves LCCPDR, LCCPT, LCCJDR and LCCJT specify additional cohesion as a function of time.

Isotropic Elastic-Plastic With Oriented Cracks model. This model is used to model brittle materials (ceramics) or porous materials, such as concrete, which undergo failure due to high tensile loads. Basically, it can be an isotropic elastoplastic or elastic material with an oriented crack. Mises flow condition:

$$\varphi = J_2 - \frac{\sigma_y^3}{3}, \quad (33)$$

where J_2 is determined based on the equation (2); plastic stress σ_y is a function of the effective plastic strain ε_{eff}^p and modulus of plastic strengthening E_p:

$$\sigma_y = \sigma_0 + E_p\varepsilon_{eff}^p, \quad (34)$$

Effective plastic deformation:

$$\varepsilon_{eff}^p = \int_0^t d\varepsilon_{eff}^p, \quad (35)$$

The pressure in this model is determined by evaluating the equations of state. In general, the fracture model with an oriented crack is based on the maximum stress criterion.

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CONCLUSIONS

Based on the analysis of common physical models of geological rocks, the Coulomb-Mohr model is best suited for describing strong rocks such as granite, gabbro, and marble. However, the geological cap and Schwer-Murray models are more flexible and universal. The specified models contain appropriate algorithms that allow obtaining a better picture of the stressed and deformed states under the conditions of dynamic destruction of materials. In the Coulomb-Mohr model, the volumetric response to deformation is purely elastic. While in the geological model or Schwer-Murray model, the volumetric response under loads is elastic in the range up to some point κ. After point κ, the system passes into the zone of plastic volumetric deformation (compaction), which is generated at a rate controlled by the law of hardening. In turn, kinematic strengthening leads to hysteresis dissipation of energy under conditions of cyclic loading. In addition, the Schwer-Murray model implements a mechanism that allows investigating the accumulation of damage. The shortcomings of the geologic model and the Schwer-Murray model include a significant set of parameters that can be determined purely experimentally.

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Аналіз поширених геологічних моделей матеріалів

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Анотація. В роботі виконаний аналіз фізичних моделей, які використовуються при дослідженні напруженого стану геологічних порід. До поширених фізичними моделей, відносяться: 1) грунту та піни; 2) псевдо – тензор; 3) геологічна; 4) Швера-Мюррея; 5) континуальна поверхня шапки; 6) Кулона-Мора; 6) з'єднаного каменю. Наведено графічне представлення опису моделі грунта та піни. На початкових етапах навантаження при малих деформаціях модель поводить себе лінійно-пружно. При збільшенні рівня деформацій перетворюється в нелінійну модель. Графічне представлення моделі псевдо-тензора відображає два режими роботи моделі в залежності від фізичних властивостей матеріалу. Геологічна модель представляє із себе один із підвидів моделі геологічної шапки і використовується при вирішенні геомеханічних проблем, а також при моделюванні таких матеріалів, як бетон. Графічне представлення геологічної моделі описується трьома кривими, функції, яких наведені в роботі. Модель Швера-Мюррея є розширеною версією геологічної моделі, яка включає вязкопластичність для розрахунку швидкісних ефектів і механіки пошкоджень. Призначена для дослідження таких матеріалів, як грунти, бетон та гірські породи. Вдосконаленою моделлю Швера-Мюррея є модель континуальної поверхні шапки (CSCM), поверхня текучості якої визначається трьома інваріантами напружень. Модель Кулона-Мора призначена для дослідження твердих елементів, товстих оболонок і частинок SPH. Використовується для представлення зв'язних або незв'язних гірських порід, ґрунтів, уламкових зцементованих гірських порід, піщаних ґрунтів та інших зернистих матеріалів. Модель з'єднаного каменю є аналогом моделей Друкера-Прагера та Кулона-Мора. Модель орієнтованої тріщини використовується для моделювання крихких матеріалів (кераміка) або пористих матеріалів, таких як бетон, і які зазнають руйнування внаслідок великих навантажень розтягу. В основному це може бути ізотропний пружнопластичний або пружний матеріал із орієнтованою тріщиною.

Ключові слова: псевдо-тензор, інваріант, границя текучості, пластична деформація, руйнування, зміцнення, бетон, гірські породи, грунт.