Computer modeling of process in earth structures of dam type

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Abstract. The article deals with computer modeling of processes in earth structures of dam type, taking into account the nonlinear properties of soil. Large vertical and planned displacements of soil in the body of the dam are formed due to changes in soil density or as a result of the development of plastic shear strains. The application of the von Mises-Botkin and Coulomb hypotheses to the development of plastic strain is investigated.

1. Introduction
Worldwide construction of underground and aboveground structures, as well as the development of ground space in seismically active regions and in the areas with developed irrigation systems and high danger of floods, stipulates for a reliable forecast of the actual state of dams, levees, underground and aboveground structures, road and railroad embankments, aircraft landing grounds in emergency situations and (or) during long-term operation.

Finite Element Method (FEM) is widely used in the analysis of embankment dams [1]. In the FEM analysis, a correct constitutive model should be identified; it is chosen for each part of the dam to model the stress-strain relationship. The zones of embankment dams have different functions. Because of this, zones usually consist of different types of soil and their stress-strain response can vary greatly. For each dam zone, the parameters entering the chosen constitutive model should have appropriate values. Generally, laboratory and/or field-testing of soil is needed as a basis to evaluate these parameters. However, a great number of dams are outdated and limited information on soil materials of the dam structures is available. Generally, it is difficult to take soil samples for testing, especially from the central impermeable part of the dam, since this can affect the dam operation and its safety. It would be useful to find a way to determine the values of the principal parameters by some non-destructive method.

The study in [2] determines the slope stability and gives a case study of the Koga embankment dam (Ethiopia). The analysis was conducted using PLAXIS 2D finite element software. The behavior of the body and the base of the dam was described using the Mohr-Coulomb criterion. Based on this study, at the end of the construction process, the resulting safety factor to analyze static stability was 1.6221. For the stationary regime, the water level was fixed at the full reservoir level (2015.25 m). The safety factor obtained for the static stability analysis was 1.6136. The analysis results showed that the safety factor for static calculations was 1.2199. Using the design standards recommended by the US Army Corps of Engineers, the British Dam Association, and the Canadian Dam Association, the slope stability analysis of the Koga embankment dam appeared to be safe under critical loads.

The structural stability of an embankment dam under static loading is investigated in [3]; a geosynthetic lining system is used as a filtration barrier, and the results are compared with the results
obtained for the same section of the embankment dam without geosynthetic lining systems. The geotechnical properties of the embankment dam were chosen so that it is stable under static conditions without any geosynthetic lining system. The results of the analysis clearly showed that the geosynthetic lining system increases the stability of the dam sections and provides a better alternative to the seepage control in the embankment dams.

In [4, 5], on the basis of integrated geo-radar and seismotomographic studies, a geomechanical model of an enclosing fill-in hydro-technical structure (a dam) was developed. The model was investigated in an elastoplastic case using computer simulation methods and resulted in determining the regularities of deformation and displacement of the structure body, and the formation of a depression curve in the body depending on the properties of its constituent soils and the level of external water load. The data obtained represent the basis for substantiating measures to reduce the risks of local destruction of enclosing dams.

The accumulated experience of embankment dams operation [6] shows that the accidents and conditions unacceptable from the point of view of normal operation can occur as a result of the development of the following processes:

- large vertical and planned displacements of soil in the dam body due to changes in the soil density or as a result of the development of plastic shear strains. Residual soil displacements can cause the crack formation on the slopes and the crest of the dam, parallel to the axis of the dam with opening up to several tens of centimeter, up to several meters deep; and the appearance of feathering fractures directed at an angle to the axis of the dam which are especially dangerous from the point of view of the anti-seepage element damage;
- violation of the monolithic character of anti-seepage element and its contact with the base, the sides of the canyon, and discharge facilities;
- erosion of the dam by a flow over the crest due to disruption of discharge facilities operation during a flood or an earthquake, caused by a sharp drop in the crest level as a result of the dam body settlement or loss of stability of its slopes, wave formation in river beds, canals, and reservoirs due to collapse of large masses of soil of coastal slopes.

The analysis of accidents and damages shows [7] that the nature of the pressure front violation of the structure can be different. It can be abrupt damage, directly related to the impact of the lowering of the dam crest level, either due to soil settlement or due to the loss of stability and the slope collapse along the sliding curve that captures the dam crest.

The finite element method is used [8] to solve the problem, taking into account the complex geometry of the region.

The study in [9] describes a technology for constructing a finite element representation of a multiply connected three-dimensional domain. The representation of the finite element configuration of the domain is described by a discrete set, which consists of a number of nodes and elements of the finite element grid, which present an ordered set of coordinates of nodes and the number of finite elements. Subdomain merging is based on the coincidence criteria for boundary nodes by determining a simple hierarchy of volumes, surfaces, lines, and points. Relabeling of nodes is performed by the frontal method, in which the nodes located at the outer edges of the structure are used as the initial front.

A method to solve a three-dimensional problem of elastoplastic deformations in a transversely isotropic body by the FEM is presented in [10]. The process of solving the problem consists of the determination of effective parameters of a transversely isotropic medium; the construction of a finite element grid of the body configuration, including the determination of the local minimum value of the bandwidth of nonzero coefficients of systems of equations by the front method; the determination of the stiffness matrix coefficients and nodal components of the load vector of the equation of state of an individual finite element according to the theory of small elastoplastic deformations for a transversely isotropic medium; the formation of a resolving symmetric band system of equations by summing all the coefficients of the equations of state by summing all the finite elements; the solution of the system
of equations of a symmetric band by the square root method; the calculation of the elastoplastic stress-strain state of the body by performing the iterative process of the initial stress method.

2. Materials and methods
The identification of the causes of the above factors necessitates a deep analysis of real properties of soil, which depend on compressibility, water permeability, contact shear resistance, and structural-phase deformation, and on the defining relationships between the components of stress and strain tensors.

The defining relationships [6, 7] and [11, 12], determined experimentally, are called the soil models and they directly depend on the nature and rate of external force application and physical and chemical composition of the soil itself, i.e., it is impossible to describe them by a single law of the stresses and strains relationship. Therefore, 4 types of regularities that dominate in practical applications are distinguished in soil mechanics as the characteristics:

1) compressibility, when calculating the base settlement;
2) water permeability in the process of predicting the water-saturated soil bases settlement rate;
3) contact resistance, when determining the ultimate strength, stability, and pressure on enclosing facilities;
4) structural-phase deformability in the process of determining soil stresses and strains; when calculating unique structures, they should be solved considering the mutual influence of all characteristic soil properties.

For some practical important classes of static problems, it is advisable to jointly solve the equations of groups (3) and (4), obtained from the relations given in [6-7] at \( \dot{e}_{int} = \dot{\sigma}_{int} = \dot{e}_{av} = \dot{\sigma}_{av} = 0 \), with equations

\[
\sigma_{int} = \Phi(e_{int}, e_{av}, \sigma_{av})
\]

and

\[
\sigma_{av} = \Psi(e_{int}, e_{av}, \sigma_{int})
\]

where

\( \sigma_{int} \) and \( e_{int} \) are the intensity of stresses and strains;

\( \sigma_{av} \) and \( e_{av} \) are the average stresses and strains.

The specific form of functions \( \Phi \) and \( \Psi \) directly depends on the real properties of soil [7, 11].

Under a single loading, the relationship between the components of the stress and strain tensors can be written as:

\[
\sigma_{ij} = \left( \sigma_{av} - \frac{2\sigma_{int}}{3e_{int}} e_{av} \right) \delta_{ij} + \frac{2\sigma_{int}}{3e_{int}} e_{ij},
\]

where

\( \sigma_{ij} \) and \( e_{ij} \) are the components of stress and strain tensors;

\( \delta_{ii} = 1, \delta_{ij} = 0 \) at \( i \neq j \);

the relationship between deformations and displacements is taken in the following form:

\[
e_{ii} = \frac{\partial u_i}{\partial x_i}, \quad e_{ij} = \frac{\partial u_i}{\partial x_j} + \frac{\partial u_j}{\partial x_i}.
\]
\[ \sum_{j=1}^{3} \frac{\partial \sigma_{ij}}{\partial x_j} + p_i = 0 \]  

(5)

are solved with relations (1)-(4), under the following boundary conditions:

\[ (\sigma_{ij} - \sigma_{ij}^*) \delta u_{ij} \Bigg|_{x_i} = 0 , \]

(6)

and the following conditions of fixing on a part of the surface:

\[ u_{ij} \bigg|_{x_i} = 0 \]

where \( \sigma_{ij}^* \) are the specified values of \( \sigma_{ij} \), \( \delta \) is the sign of the variation.

When solving the problems of equilibrium of earth structures, the von Mises-Botkin models were tested to determine function \( \Phi \) [11]:

\[ \tau_{int} = G \gamma_{int} - \tan \psi \sigma_{av} , \]

(7)

and the Coulomb-Mohr models to determine function \( \Psi \):

\[ \tau_{int} = c - \tan \varphi \sigma_{av} \]

(8)

where:

\[ \tau_{int} \] and \( \gamma_{int} \) are the intensities of shear stresses and angular strains;
\[ c \] is the coefficient of cohesion;
\[ G \] is the shear modulus, \( \varphi \) and \( \psi \) are the angles of internal friction on the sliding surface and on the octahedral plane, respectively.

In the limit state problems, the following relations are used, instead of (7) and (8):

\[ \tau_{int} = \tau_s - \tan \psi_s \sigma_{av} \]

(9)

and

\[ \tau_{int} = c - \tan \varphi_s \sigma_{av} \]

(10)

where

\[ \tau_s = G \gamma_s , \quad \gamma_s \] is the shear yield strain;
\[ \varphi_s \] and \( \psi_s \) are the limit values of \( \varphi \) and \( \psi \).

One of the central issues in the earth dam calculation considering plastic strain is the choice of the defining equations of soil that characterize the stress-strain relation [15]:

\[ \begin{align*}
\sigma_{11} &= K (g_1 e_{11} + g_2 e_{22} + g_2 e_{33}) ; \\
\sigma_{22} &= K (g_2 e_{11} + g_1 e_{22} + g_2 e_{33}) ; \\
\sigma_{33} &= K (g_2 e_{11} + g_2 e_{22} + g_1 e_{33}) ; \\
\sigma_{12} &= K g_3 e_{12} ; \\
\sigma_{13} &= K g_3 e_{13} ; \\
\sigma_{23} &= K g_3 e_{23} ,
\end{align*} \]

(11)

where

\[ K = E / (3(1 - 2\mu)) \] is the modulus of volumetric compression;
\[ g_1 = \theta + 2\beta / 3 ; \quad g_2 = \theta - \beta / 3 ; \quad g_3 = \beta / 2 ; \]
\[ \theta = 1 \] at \( \sigma_{av} = 3Ke_{av} ; \quad \theta = f_m + h_m / e_{av} ; \quad (m = \overline{m_0}) \) at \( \sigma_{av} = 3\Phi (e_{av}) , \)

where \( \Phi \) is replaced by splines \( \Phi = f_m e_{av} + h_m ; \)
in the case of considering dilatancy $e_{av,cd}$: \( \theta = (1 + e_{av,cd} / e_{av}) f_m + h_m / e_{av} \); the value of $e_{av,cd}$ is determined from the relation, where $\delta_1 = 3Ge_{int}$, $\delta_2 = \tau \gamma \mu$ holds for the von Mises-Botkin model; $\delta_1 = c$, $\delta_2 = \tau \gamma \phi$ holds for the Coulomb-Mohr model.

The coefficient $\beta$ in the von Mises-Botkin model is defined as $\beta = 3(1 - 2\mu)/(1 + \mu) - \sqrt{2}\tau \gamma \mu \theta (e_{av} / e_{int})$, and in the Coulomb-Mohr model it is defined as $\beta = \sqrt{2}(c / (Ke_{int}) - \tau \gamma \phi \theta (e_{av} / e_{int})$.

An account for the nonlinearity effect of $\sigma_{av}$ and $e_{av}$ relation increases the stress intensity values by 37.4%.

3. Results and discussion

In the test examples given below, the nature of the stress-strain state redistribution in the Tupolang embankment dam (Uzbekistan) was studied (the height of the dam is $h = 180$ m; the length and the width of the dam in the base are $712$ m and $380$ m, respectively; the width of the crest is $b = 16$ m, the width in the base cross section is $L_0 = 250$ m, the upper and lower slopes are taken as the ratios 1.0:2.0 and 1.0:1.9; the core is from loam; the retaining prisms are from pebble; $\sigma_{av} = 5$ kg/cm$^2$; $E = 9798$ kg/cm$^2$; $\mu = 0.3633$; $c = 0.3$ kg/cm$^2$; $\gamma_i = 2.2$ kg/cm$^3$ [15]) depending on the models used: the von Mises-Botkin model at $\theta = 1$ and $\theta = f_m + h_m / e_{av}$ and an option of the Coulomb-Mohr model at $\theta = 1$.

Figures 1 and 2 show graphs of stress intensity values according to von Mises with and without considering nonlinearity (the order of the system of equations is $n = 7488$ when using quadratic isoparametric hexagonal finite elements). An account for the nonlinearity effect of $\sigma_{av}$ and $e_{av}$ relation increases the stress intensity values by 37.4%.

**Figure 1.** Graph of stress intensity according to von Mises.
The results obtained using von Mises-Botkin and Coulomb-Mohr models (figures 3 and 4) at $\theta = 1$ differ substantially, since the Coulomb-Mohr model uses parameters corresponding to the limit values of soil. Fig. 4 shows the values of $\sigma_{av}$. The character of the FEM convergence is determined depending on a shape of the elements (a pyramid with 4 and 10 nodes, an isoparametric hexagon with 8 and 20 nodes) and an implicit scheme of an iterative process. When using elements in the form of a pyramid (4 nodes, $n = 1998$) and a hexagon (8 nodes, $n = 1400$), the difference in the maximum values of $\sigma_{int}$ in the case of the von Mises-Botkin model ($\theta = 1$) is 3.7%.

Figure 2. Graph of stress intensity according to von Mises, taking into account nonlinearity.

Figure 3. Graph of average stress according to the von Mises-Botkin model.
Figure 4. Graph of average stress according to the Coulomb-Mohr model.

In the case of the von Mises-Botkin model, the accuracy $\varepsilon_{om} = 0.01$ is achieved in three iterations, and in the case of the Coulomb-Mohr model, 3-4 times more iterations are required to achieve this accuracy. This is due to the fact that the Coulomb-Mohr models use limit values of soil.

4. Conclusion
The article deals with computer modeling of processes in earth structures of dam type, taking into account the nonlinear properties of soil according to the von Mises-Botkin and Coulomb-Mohr models. On the basis of the software developed and the computational experiment conducted, the nature of the stress-strain state redistribution in the Tupolang embankment dam (Uzbekistan) was studied. It was determined that an account for the nonlinearity effect of $\sigma_{av}$ and $e_{av}$ relation increases the stress intensity values by 37.4%.

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