EFFECT OF THE PORE WATER PRESSURE ON THE STRESS-STRAIN BEHAVIOR OF EARTH DAMS

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ABSTRACT

Results of a representative study on the time-depending stress-strain behaviour evaluation (consolidation analysis) are presented. The basic constitutive equations, describing the rheological processes of the clay skeleton and generalized differential equation of two-dimensional consolidation are given. Some results of the computations for determining the pore water pressure distribution and settlement during construction and operation of a high dam and ultimate stress-strain behaviour of the dam subgrade are analyzed. The computations are performed applying computer codes, developed by the author, using the solutions of the two-dimensional consolidation of multi-phase clayey soils taking into account the non-linear strain characteristics of the clay, deformability of the pore liquid (due to the incomplete saturation) and the creeping of the soil skeleton.

INTRODUCTION

The high dams, composed of clay core and rock fill, are highly responsible geotechnical structures. Assessment of the stability of such earth dams often requires a comprehensive analysis in order to determine the stress distribution and deformation induced in the dam by the static and time-depending loads. Built-up of different granular materials, the dams form a complicate stress-strain behaviour that has been changed during construction and operation. In the cases of clay embankments one of the important factors of the dam stability is the evaluation of the pore water pressure distribution and its variation in time. This problem is usually solved on the base of the theory of consolidation.

At present, the theory of consolidation is developed on the base of two general principles - the principle of effective stresses (Terzaghi, 1923) and principle of volume forces (Florin, 1939, Biot, 1941). Both principles consider the interaction between pore water and soil skeleton during consolidation on different way. The filtration process is study, and two tensors (for liquid and hard phases) characterize the soil stress-strain behaviour. The Terzaghi's principle of effective stress is based on the following assumptions: the pore water filtration is described according to Darcy's law; the water and air mixture could be considered as deformable; the sum of the total normal stress is constant during the consolidation process. The other principle, proposed by Florin (1939), accepts that during consolidation process the volume forces are formed as a result of some interaction between the hard and liquid phases. A great numbers of contributions to the consolidation theory, based on both general principles, have been made by means introducing different soil models. It is impossible to enumerate all theoretical contributions in this field, but we would mention only two capital books (in Russian) - Florin (1961) and Zaretsky (1967). A comprehensive analysis on the consolidation problem was made by Balasubramian & Brenner (1981).

Clayey soils, used for earth embankments as a part of geotechnical structures, have been compacted during construction and pore water pressure will generate only under great loads. The creeping properties of the soil skeleton must be taken into account for stress-strain behaviour evaluation.
The aim of this paper is, therefore, to present an approach for determining the stress-strain-time behaviour of the cohesive soils, taking into account their specific features, such as multiphase nature (non-full saturation), creeping of soil skeleton and filtration of pore water. Comparing the results of the theoretical and experimental study (Germanov, 1988), a conclusion has been made, that this model could be applied for more exact evaluation of the stresses and strains of high clay embankments.

**BASIC CONSTITUTIVE EQUATIONS MODELING THE CLAY BEHAVIOR**

The analysis of the stress-strain-time of soil masses is a basic problem of the applied Geomechanics in the Civil Engineering. In the cases of non-full saturated clays the soil massive could be considered as multi-phase medium and their stress-strain behaviour being accompanied by two simultaneously, rheological processes: filtration and creep of the soil. It is supposed that, due to incomplete saturation, the fluid is linearly deformable, while the deformation of the soil skeleton could be presented according to the theory of linear creep. The equation that describes the skeleton conditions, according to Arutjunan (1952) and Florin (1961) acquires the following form:

\[
e_0 - e(t) = \frac{\vartheta(t)}{1 + 2K_0} m_v(t, t) - \frac{1}{1 + K_0} \int_{\tau}^{t} \vartheta(\tau) m_v(t, \tau) d\tau \; ; \quad (1)
\]

where: \( e_0 \) and \( e(t) \) are the initial and variable over time void ratios;
\( \vartheta(t) \) is the sum of normal effective stresses at the fixed point of the soil massive;
\( K_0 \) is the coefficient of the lateral pressure at rest;
\( m_v(t, \tau) \) is the generalized coefficient of volume strain.

\[
m_v(t, \tau) = m_0(\tau) + \varphi(\tau)[1 - \exp(-\eta(t - \tau))]; \quad (2)
\]

\( m_0(\tau) \) is the coefficient of instantaneous strain (linear compressibility);
\( \varphi(\tau) \) is the function of ageing (tixotropic strengthening) of the soil skeleton.

\[
\varphi(\tau) = m_l + \frac{m_h}{\tau}; \quad (3)
\]

\( m_l \) is the coefficient of volume creep strain (secondary compression);
\( \eta \) - the parameter of creeping speed;
\( m_h \) - the coefficient of "ageing" strain of the soil skeleton;
\( \tau \) - the parameter of the soil skeleton age (the previous stressed condition).

The methods for determining the coefficients of volume strains and the creep parameters are developed by the author in Germanov (1978, 1983).

Examining the pore water and the air dissolved into the water as a component which is deformed as a results of the air bubbles compression (Tsytovich & Ter-Martirosyan, 1981), the following equation, describing the liquid deformation, is used:

\[
\frac{1}{\rho_w} \frac{d\rho_w}{du_w} = \frac{1 - S_r}{p_a} = m_w; \quad (4)
\]
where: \( \rho_w \) is the water density;
\( u_w \) - the pore pressure (stress of the pore liquid, or neutral stress);
\( m_v \) - the coefficient of linear strain of the pore liquid;
\( S_r \) - the degree of saturation;
\( p_a \) - atmospheric pressure.

Assuming that the fluid filtration is according to Darcy's law, the function \( u_w(t,x,y) \) for two-dimensional consolidation, is determined by solution of the following differential equation:

\[
a_1 \frac{\partial^2 u_w}{\partial t^2} + f_1(t) \frac{\partial u_w}{\partial t} = C_v \left( (\Delta u_w) - \eta \Delta u_w \right) - f_2(t) \frac{\partial}{\partial t} \theta_1(t) - a_2 \frac{\partial^2}{\partial t} \theta_1(t),
\]

where: \( a_1 = A_w + 2a_2; \) \( a_2 = 1/(1+K_0) \);

\( f_1(t) = \eta A_w + 2f_2(t); \)
\( f_2(t) = \eta a_2[1+f_3(t)]; \)
\( f_3(t) = A_i + A_w t; \)

\[
\Delta u = \frac{\partial^2 u_w}{\partial y^2} + \frac{\partial^2 u_w}{\partial z^2}; \quad \theta_1(t) = \sigma_i(t) + \sigma_z(t);
\]

\[
A_w = \frac{e_o m_w}{m_b}; \quad A_i = \frac{m_i}{m_o}; \quad C_v = \frac{(1+e_o)k_f}{m_o \gamma_w}.
\]

\( \sigma_i(t) \) and \( \sigma_z(t) \) are the total normal stresses which would be accepted:
\( \sigma_i(t) = \text{const}; \sigma_z(t) = \text{const} \) - when the period of operation is considered;
\( \sigma_i(t) = a_i t; \sigma_z(t) = a_z t; \) - variation with a constant velocity, for the construction period;
\( \sigma_i(t) = \sigma_{i0} \sin \omega t; \sigma_z(t) = \sigma_{z0} \sin \omega t \) - variation under cyclic loads (machine foundations or earthquake motions), \( \sigma_{i0} \) and \( \sigma_{z0} \) are static components of the total stresses;
\( \omega \) - natural frequency of the ground;
\( k_f \) - coefficient of permeability, accepted to be equal in directions \( y \) and \( z \); 
\( \gamma_w \) - unit weight of the water.

Equation (5) we consider as more generalized which could be applied for determination of the pore water pressure generation and dissipation in soil massives composed by different clayey soils. For example, if we examine low plastic clay, the ageing properties could be ignored, and (5) is then simpler. In the cases of full saturated and very soft soils, the creep properties are slightly expressed, then the equation (5) describes stress behaviour according to the Terzaghi's theory.

The numerical solution of equation (5), based on the method of Riz-Galerkin, satisfying exactly the initial condition and approximately the boundary conditions, is presented by Germanov and all (1996).
DETERMINATION OF SOIL PARAMETERS

The author (Germanov, 1978, 1983) has developed methods for evaluation of the coefficients of volume strain and creep parameters using special and standard odometers. The results from compression tests in drained and undrained conditions are used. Part of the parameters could be evaluated by the "time-settlement" curves and the other - by the "time-pore pressure" curves.

A great number of clayey soil samples taken from different regions are tested. After the analysis of the results obtained, some correlation formulas for determination of the consolidation parameters as function of the void ratio and the index plasticity are suggested (Germanov & Soffev, 1998). The first correlation could be used for evaluation of nonlinear stress-strain behaviour, and the other one (using index plasticity) could be applied for different type of cohesive soils.

Comparing the experimental results of different cohesive soils in undisturbed and disturbed (compacted) conditions a conclusion was made that the ageing parameters have more significant values for the undisturbed samples of high plastic clays. On this reason, the computations for evaluation of the pore water pressure in high clay embankments are performed neglecting the ageing parameters.

A computer code for determining the coefficients of volume strain and creeping parameters is developed. The results from laboratory tests are used as input data. Corrective coefficients are applied for harmonization of the laboratory and in-situ characteristics.

CONSOLIDATION ANALYSIS OF HIGH DAMS

Consolidation analysis, i.e., clay core pore water pressure and dam settlement, includes computations for the assessment of the stress-strain-time behaviour during construction and operation. The computations of the pore water pressure generation and dam settlement are carried out using the computer codes developed by the author.

The following preconditions are used for the purpose:

- During the construction the total normal stresses increase uniformly, and the erection of the dam shoulders is to follow the laying of the clay core;
- During the operation, the impoundment is accomplished after the end of the construction;
- The variation of the soil characteristics, after the compaction during construction, is taken into account applying step by step computation.

Several high dam projects (situated in Bulgaria, Syria, Algeria) are performed by using the above mentioned computer codes. Some results of the "Malkien Dam" project - Syria are given below. The dam project includes a clay core with maximum height of 80m. The other feature of the project is that the dam is situated on a deformable subgrade composed of several sand and sandy-clay layers. In this conditions the stress-strain behaviour of the dam is evaluated taking into account the subgrade deformability. The design geotechnical parameters for the clay core consolidation study are given in Table 1. Taking into account that during the filling the clay will be compacted, the variation of the basic characteristics along the dam height is evaluated.

The numerators in the columns No 5, 6 and 7 of Table 1 present the compression characteristics, and denominators - the in situ characteristics. The compression characteristics are used for the pore water pressure computation during the constriction period, and the in situ characteristics - for the consolidation analysis (pore water pressure and settlement) during the operation.
The computations are carried out under the following conditions:

- The dam embankment is to be laid over a period of 3 years with 4-month interruptions each year. The erection of the dam shoulders is to follow the laying of the clay core.
- The impoundment is to be accomplished one year after the end of the construction.

### Table 1. Geotechnical parameters for different heights of the clay core

<table>
<thead>
<tr>
<th>$H$ (m)</th>
<th>$e$</th>
<th>$\gamma_a$ (kN/m³)</th>
<th>$k_f$ (10⁻⁴ cm/d)</th>
<th>$M/E_0$ (MPa)</th>
<th>$m_0$ (1/MPa)</th>
<th>$m_f$ (1/MPa)</th>
<th>$\eta$ (1/d)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td>6</td>
<td>7</td>
<td>8</td>
</tr>
<tr>
<td>0.0</td>
<td>0.695</td>
<td>19.15</td>
<td>0.468</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.0411</td>
</tr>
<tr>
<td>10</td>
<td>0.659</td>
<td>19.35</td>
<td>0.367</td>
<td>9.1</td>
<td>14.6</td>
<td>0.0328</td>
<td>0.1495</td>
</tr>
<tr>
<td>20</td>
<td>0.622</td>
<td>19.50</td>
<td>0.285</td>
<td>9.9</td>
<td>15.8</td>
<td>0.0295</td>
<td>0.1343</td>
</tr>
<tr>
<td>30</td>
<td>0.585</td>
<td>19.71</td>
<td>0.230</td>
<td>10.7</td>
<td>17.1</td>
<td>0.0266</td>
<td>0.1214</td>
</tr>
<tr>
<td>40</td>
<td>0.553</td>
<td>19.79</td>
<td>0.180</td>
<td>12.0</td>
<td>19.2</td>
<td>0.0233</td>
<td>0.1061</td>
</tr>
<tr>
<td>50</td>
<td>0.526</td>
<td>19.83</td>
<td>0.145</td>
<td>13.0</td>
<td>20.8</td>
<td>0.0211</td>
<td>0.0963</td>
</tr>
<tr>
<td>60</td>
<td>0.502</td>
<td>19.90</td>
<td>0.105</td>
<td>16.8</td>
<td>26.9</td>
<td>0.0169</td>
<td>0.0769</td>
</tr>
<tr>
<td>70</td>
<td>0.483</td>
<td>20.13</td>
<td>0.095</td>
<td>18.6</td>
<td>29.8</td>
<td>0.0148</td>
<td>0.0676</td>
</tr>
<tr>
<td>80</td>
<td>0.466</td>
<td>20.21</td>
<td>0.084</td>
<td>19.0</td>
<td>30.4</td>
<td>0.0139</td>
<td>0.0633</td>
</tr>
</tbody>
</table>

The clay core pore water pressure distribution for the highest cross section at the end of the construction period and during the operation is presented by isochrones (lines joining the points with the same pore water pressure values) on fig.1.

Bearing in mind the clay core dimensions (bottom width approximately equal to the height) and the short time of the dam erection (3 years), the pore water pressure values at the end of the construction period are comparatively high. For example, the pore water pressure coefficients, presented as a ratio $u_w/\sigma_z$ ($u_w$ - pore water pressure; $\sigma_z = \gamma z$ - total vertical stress), decrease from 0.45 (for the points situated on the core axis near the bottom) - to 0.10 (for the points situated close to the crest and the core slopes). The comparatively high pore water pressure values computed at the end of the construction period and the slow pore water pressure dissipation (because of the long filtration path) influence on the pore water pressure distribution during the operation with the full reservoir. For the higher cross sections, even after 15 years of operation with full reservoir, the pore water pressure isochrones do not reach the filtration pressure isochrones.

In order to evaluate the final (stabilized) dam crest settlement and its variation in time, a clay core and subgrade consolidation analysis is performed.
The presence of soils, such as sand of various grain size, in some places argillaceous, and marly clays presumes a time dependent subgrade settlement under the dam loading. Some generalized results of the computations of the settlements along the main dam axis are given in Table 2.

Fig. 1. Pore water pressure isochrones in the clay core at the end of construction and during operation

The degree of consolidation, i.e. the ratio of the final (stabilized) settlement to the settlement at the end of the construction period is different for the cross sections under consideration (from 68% for cross section V, to 93% for cross section II).

The predicted dam crest additional height, presented as a settlement during the operation, has different values for each cross section as well (10.0cm for cross section II and 36.9cm for cross section III).

Bearing in mind the short planned construction period, the stabilization of the clay core strains, i.e. the attainment of the expected final settlements values, can continue for more than 10 years after the completion of the dam construction. A prolongation of the construction period would lead to the attainment of the greater part of the predicted settlement during construction. Thus the predicted dam crest additional height would be smaller.
The computation results indicate non uniform subgrade settlements. For example: The degree of subgrade consolidation at the end of the construction period reaches 45 - 50% for cross sections III and IV; 68% for cross section I; and 100% for cross section V. Therefore, an additional consolidation analysis is performed for the cross sections I and II, where the subgrade layers with different deformability characteristics are situated. The aim of the computations is an assessment of the non uniform settlement effect on the grouting gallery strains. The settlement difference of both cross sections is 9.8 cm, which means 0.53% relative strain. These results should be considered carefully because this non uniform strain is applicable for two points only at a distance of 18.5 m. It is obvious that it will be developed uniformly along the entire distance. Nevertheless compensation joints must be provided in the grouting gallery for cracking security.

Table 2. Dam crest settlement taking into account the subgrade deformability

<table>
<thead>
<tr>
<th>Description</th>
<th>Cross sections</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>I</td>
<td>II</td>
<td>III</td>
<td>IV</td>
<td>V</td>
</tr>
<tr>
<td>Height of the cross sections, m</td>
<td>32.8</td>
<td>44.4</td>
<td>74.0</td>
<td>63.9</td>
<td>32.9</td>
</tr>
<tr>
<td>Type of the settlement, cm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Final (stabilized) settlement</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>• embankment</td>
<td>70.7</td>
<td>120.3</td>
<td>280.3</td>
<td>233.5</td>
<td>70.7</td>
</tr>
<tr>
<td>• subgrade</td>
<td>16.6</td>
<td>26.4</td>
<td>43.0</td>
<td>38.3</td>
<td>11.3</td>
</tr>
<tr>
<td>• total</td>
<td>87.3</td>
<td>146.8</td>
<td>323.3</td>
<td>271.8</td>
<td>82.0</td>
</tr>
<tr>
<td>Settlement during the construction period</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>• embankment</td>
<td>44.5</td>
<td>116.1</td>
<td>266.7</td>
<td>221.7</td>
<td>44.5</td>
</tr>
<tr>
<td>• subgrade</td>
<td>14.8</td>
<td>20.7</td>
<td>19.7</td>
<td>18.5</td>
<td>11.3</td>
</tr>
<tr>
<td>• total</td>
<td>59.3</td>
<td>136.8</td>
<td>286.4</td>
<td>240.2</td>
<td>55.8</td>
</tr>
<tr>
<td>Settlement during the operation</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>• embankment</td>
<td>26.2</td>
<td>5.7</td>
<td>13.6</td>
<td>11.8</td>
<td>26.2</td>
</tr>
<tr>
<td>• subgrade</td>
<td>1.8</td>
<td>4.3</td>
<td>23.3</td>
<td>19.8</td>
<td>0.0</td>
</tr>
<tr>
<td>• total</td>
<td>28.0</td>
<td>10.0</td>
<td>36.9</td>
<td>31.6</td>
<td>26.2</td>
</tr>
<tr>
<td>Residual settlement after 5 years of operation</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>• embankment</td>
<td>2.8</td>
<td>3.5</td>
<td>12.8</td>
<td>10.6</td>
<td>2.0</td>
</tr>
<tr>
<td>• subgrade</td>
<td>1.1</td>
<td>1.2</td>
<td>9.6</td>
<td>9.9</td>
<td>0.0</td>
</tr>
<tr>
<td>• total</td>
<td>3.9</td>
<td>4.7</td>
<td>22.4</td>
<td>20.5</td>
<td>2.0</td>
</tr>
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<td>Residual settlement after 10 years of operation</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
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<td>• embankment</td>
<td>1.5</td>
<td>3.1</td>
<td>12.0</td>
<td>10.0</td>
<td>1.6</td>
</tr>
<tr>
<td>• subgrade</td>
<td>1.0</td>
<td>0.0</td>
<td>0.7</td>
<td>5.2</td>
<td>0.0</td>
</tr>
<tr>
<td>• total</td>
<td>2.5</td>
<td>3.1</td>
<td>12.7</td>
<td>15.2</td>
<td>1.6</td>
</tr>
</tbody>
</table>

An additional (beyond the design assignment) study, of the assessment of the final (stabilized) dam settlements (including rock embankment), is carried out. The computations are performed assuming that the rock embankment settlements under the overburden pressure
are developed at the end of the construction period. Therefore, the results obtained could be used for an evaluation of the extra embankment quantity. An approximate computation shows that the extra dam embankment is about 1.3 - 3.0% for the entire volume.

ULTIMATE STRESS-STRAIN BEHAVIOUR OF THE EARTH DAM SUBGRADE

The solution of the equation (5) allows determining the effective normal stress on any point of the soil massive. Then using Mohr-Coulomb equation for ultimate equilibrium (6) the dimensions of the so-called "plastic zones" in unconsolidated condition could be evaluate.

\[
\theta_{cr} = \arcsin \frac{\sqrt{\left(\sigma_z - \sigma_y\right)^2 + 4\tau_{zy}^2}}{\sigma_z + \sigma_y + 2c \cot \varphi},
\]  

(6)

The zones of ultimate equilibrium are defined by the condition \( \theta_{cr} < \varphi' \) (\( \varphi' \)- effective angle of internal friction). If these zones are relatively large, there could be a possible lost of the soil massive bearing capacity.

![Lithological profile and loosing of the subgrade bearing capacity under embankment](image)

Fig.2. Lithological profile and loosing of the subgrade bearing capacity under embankment

The application of this approach for dam stability evaluation is illustrated by the analysis of the crash of a highway embankment, built up on soft soils. The lithological profile is shown on Fig.2. The embankment was filled for two weeks only. In spite of the relatively small surface load, the pore water pressure increased rapidly and the stress behaviour of the subgrade is considerably changed (fig.3). Under these conditions, the values of the critical angle of friction (\( \theta_{cr} \)) decrease, which led to softening (liquefaction) of the subgrade and formation of large plastic zones (fig2).
CONCLUSION

The results of the above-considered computations show that when clayey soils are used for high embankments the pore water pressure has essential effect on the stress-strain behaviour of the soil massives. The constitutive equations and the method used for consolidation analysis allow a more exact evaluation of the stress distribution and the dam settlement during construction and operation. It is also shown that in the cases of very fast construction and when the subgrade is composed of soft soils, the pore water pressure generation can lead to loosing of the dam stability.

REFERENCES


