Estimation of hydraulic structures safety by comparison of strength and stability theories

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Abstract. Hydraulic structures being structures of first class require meeting the safety operation conditions. To secure safety operation conditions of earthfill dams it is important to evaluate their coefficients of stability and strength factor by calculation of mode of deformation of dam body. To complete these calculations the dam body soil actual state observation has to be carried out by experiments. Based on experimental data the mode of deformation of dam body is revealed by software modeling investigations, and safety operation conditions are evaluated. The mode of deformation is defined by nonlinear equations, which specify elasto-plastic state of dam body soil. Hydraulic structures safety and stability evaluation methods are elaborated in case of dynamic impact.

1. Introduction

Hydraulic structures being structures of first class require meeting the safety operation conditions. Periodic complex checkup of technical state and based on its analyses specification of the given structure’s safe operation evaluation is drawn up. Within the framework of the specification engineering estimates are given on technical state of the structure under study and conclusions are drawn for its further safe operation.

Often based on these conclusions implementations of a number of measures are planned which create conditions for further safe operation of the structure.

2. Statement of the problem

Accurate and optimal assessment of dams’ strength and stability is especially important in the design stage. More precise estimation of external influences and by their employing accurate estimation of strength and stability of structures create guarantees for further safe operation of dams.

However it is necessary to check again safety and stability of already erected and operating hydraulic structures. It especially is conditioned by time-dependent variation of physical-and-mechanical properties of soils. External influences acting on the given structure are also changed with the passage
of time. As a characteristic example is seismic conditions change represented to existing dams foundations resulting in increase of horizontal acceleration in the entire area of the Republic of Armenia.

3. Methodology
Estimation of safety factor and stability margin is possible only after obtaining physical-and-mechanical properties of the dam body soil. To this end it is necessary that during monitoring, as a mandatory condition, laboratory analyses [1, 2] for obtaining physical-and-mechanical properties of tested soil specimen taken from the dam body must be borne in mind. Naturally, the period and necessity of such analyses depend on the characteristics of the given structure. Based on laboratory tests it is necessary to develop a mathematical model of stress-strain behavior of the soil and use it as a tool for carrying out computer-based experimental research to determine the field of deformation mode of the dam body. Having the clear picture of the stress-strain field of the dam body its safety factor and stability margin are computed.

To carry out research on stress-strain behavior of ground dams equations [3] of linear and non-linear elasto-plastic deformation theories are used. Dam slopes stability calculations are made making use of equations of solid bodies boundary equilibrium. For calculation of the dam safety and stability margins there should exist a limit for which the safety and stability of a structure are satisfied, that is one kind of calculations serve as a base for other kinds of calculations. As for conclusions, they should be interrelated.

It is well known that there are not clearly specified mathematical models designed for accurate estimation of hydraulic structures’ stability, therefore stability problems of hydraulic structures have not yet been completely studied. It means that we have only approximate stability estimation of dams based on a plane problem, while it is necessary to checkup stability of the dam based on a three-dimensional problem. The latter is connected to a number of practical and theoretical complexities. Even in this case the accepted approximations do not ensure satisfactory accuracy.

As stated above the accurate estimation of hydraulic structures’ stress-strain behavior can be a strong base for accurate and reliable estimation of their stability.

4. Results
Theoretical footing of this method in case of the plane problem employing finite elements principles have already been used in practice. This method enables checking up dam slopes stability for different values of porous pressure ($u$) and seismic loads. In this case using finite elements both the stress-strain field and the field of filtration gradients and porous pressure are calculated in the dam body under static and dynamic loads and then they are transferred to the sliding mass.

With this end in view the obtained values are averaged out at finite elements grid nodes and after that using Mohr relationships at the center of each layer’s footing normal and tangent stresses (1,2) are obtained. Supporting forces appearing on the slip surface are obtained by Coulomb’s law [3]

$$\sigma_n = \frac{\sigma_x + \sigma_y}{2} + \frac{\sigma_x - \sigma_y}{2} \cos 2 \theta + \tau_{xy} \sin 2 \theta,$$

$$\tau_m = \tau_{xy} \cos 2 \theta - \frac{\sigma_x - \sigma_y}{2} \sin 2 \theta,$$

$$S_r = \tau \beta = c' + (\sigma_n - u) tg \phi', S_m = \beta \tau_m,$$

In case of finite elements employing [4, 5] we have the dam slope stability factor $K_{FEM} = \frac{\sum S_r}{\sum S_m}$.

The same approach can be employed for three-dimensional problem which for the continuum should be solved if the given law of the material is known. Laws of materials are obtained by laboratory tests. The more accurate results are obtained by stabilometric tests. Boundary surfaces of materials are also obtained by stabilometric tests [6].

Accurate estimation of stability of hydraulic structures is especially important from seismic impacts considerations. Toward this end it is necessary to solve a dynamic equilibrium equation in matrix form
\[ [M][\ddot{a}] + [D][\dot{a}] + [K][a] = \{F\}, \text{ where } [M] = \int_B \rho \{N\}^T \{N\} dS \text{ is the matrix of the mass, } \rho \text{ is the specific mass, } [D] = \alpha [M] + \beta [K] \text{ is the damping matrix representing resistance of the material (extinction forces), } [K] = \int_B [B]^T [C][B] dS \text{ is the stiffness matrix, } \{F\} = \{F_p\} + \{F_i\} + \{F_n\} + \{F_g\} \text{ is the force vector at the node, } \{F_p\} \text{ is gravitation force, } \{\ddot{a}\} \text{ is the acceleration vector at the node, } \{\dot{a}\} \text{ is the velocity vector at the node, and } \{a\} \text{ is the node displacement matrix [7].} \]

In case of elastic linear problems the constructive matrix ([C]) is calculated according to Eq. (4) [8, 9]. Since real soils have non-linear properties, then the constructive matrix is calculated for each loading stage.

\[
[C] = \frac{E}{(1+\nu)(1-2\nu)} \begin{bmatrix}
1 - \nu & \nu & 0 \\
\nu & 1 - \nu & 0 \\
0 & 0 & 1 - 2\nu \\
0 & 0 & \frac{1 - 2\nu}{2}
\end{bmatrix}.
\]

Dynamic (cyclic) loading increases porous pressure leading to the effective stress-strain state change of the body. That change is conditioned by respective change of mechanical properties of the material, i.e. the strength of soil material in case of dynamic loading acquires kinematic properties.

The stress-strain state is approximated with high degree of accuracy by a hyperbolic curve (5) given by the initial shear modulus (\(G_{\text{max}}\)), which corresponds to small deformations and shear stress asymptote to which tend maximum values of shear stresses [10, 11]. When in case of alternate loading a hysteresis deviation occurs, then the dependency curve becomes hyperbolic. Loading, unloading, and even reloading are carried out by the same hyperbolic curve. The advantage of this model is in that only a few parameters such as \(G_{\text{max}}\), \(\phi\) and \(C\) are required for carrying out calculations, as for the mechanism it is quite simple [9, 12]:

\[
\sigma_1 - \sigma_3 = \frac{E_1}{a + b\varepsilon_1},
\]

For accurate calculation of a non-linear model an assumption (MES) is made which enables to calculate dependency of porous pressure on the number of repeated cycles. It is assumed that loading is made in drainless conditions, therefore the increase of porous pressure is conditioned by the increase of volumetric deformation of dry soil. The increase of the porous pressure is \(\Delta u = E_r \Delta \varepsilon_{vd}\), where \(\Delta u\) is the increase of porous pressure, \(\Delta \varepsilon_{vd}\) is the volumetric deformation increase of which dependency on the number of cycles and is determined by laboratory experiments. \(E_r\) is the module of reconstruction volumetric deformation which depends on the stress and is determined experimentally.

Such assumption enables in case of Coulomb-Mohr boundary surface on the hyperbolic model to provide displacement of the hydrostatic axis in the field of the principal stresses both for plane and three-dimensional problems.

If at any point of a soil structure the soil does not resist to shear stresses then one can conclude that there is no general destruction, possibly due to plastic deformations at those points redistribution of stress-strain state occurs in the body. At the expense of such redistribution the neighboring points undertake external load ensuring the stability of the entire structure. Hence, calculations of strength and stability of soil structures should be considered in a unity. Here static and dynamic loads as well as different theories applicability limitations should be taken into account.

Otherwise speaking, if stability calculations based on the boundary equilibrium condition immediately give a picture on the dam slope stability but does not take into account stresses redistribution due to deformations development in the material, then the study of stress-strain behavior of the dam soil material comprehensively characterizes distribution of stresses and deformations in the body but does not give a clear idea on the dam and dam slopes stability. We can arrive to a conclusion that in the process of soil dams design or study these two problems need complex solution, providing strength, stability, and economic profit of dams.
The above-mentioned problem is especially relevant for the Sarsang earthfill dams, which is the highest ground dam on the continent of Europe, with a height of 127 m and a width of 560 m at the top. It is built on a site with complex engineering-geological conditions and stores 580 million m$^3$ of water. The area where the dam is built is an active seismic zone. The dam is located in the Arshakha-Azerbaijani conflict zone; in this case, incorrect professional assessments during the development of exploration declarations can lead to a dam crash, which can be politicized for mercenary purposes.

Taking into consideration all these circumstances and the fact that the dam was built in the 1970s, the Sarsang Dam was chosen as an example for calculations to illustrate the above logic more objectively.

Thus, after the devastating 1988 Spitak earthquake, the seismic requirements for existing structures in the region were significantly tightened, which resulted in a nearly two-fold increase in horizontal acceleration values. The combination of the numerical results of the mathematical models of these two variables will allow for quantitative assessments of the basic criteria for dam strength and stability.

Sarsang dam shaft stability tests in case of special loading are carried out by two different methods.

In the first case, the circular method of estimating the stability of the slopes was used for the stable determination of the slopes, and the calculations were performed according to the Morgenstein-Price method, and the horizontal seismic load epurium was adopted as rectangular according to the height of the dam. The value of horizontal acceleration was assumed to be 0.40g. The mechanical properties of the dam body soils were verified by laboratory examination of samples of different depths.

Second, the differential equation is solved in the case of a flat problem by applying finite elements, where the dependence of the ground deformations and stresses is accepted by Hyperbolic law, and the coefficient of the second term is not neglected. Taken as a special load in 1971, the magnitude of the earthquake in San Fernando on February 9 (magnitude 0.43), the magnitude of which is estimated at 6.6 on the Richter scale. In this case, the ground-facing parameter was obtained using empirical formulas.

![Figure 1. Calculation section of Sarsang dam wall buffer stability test.](image)

To determine the Poisson coefficient of the ground slip modulus, empirical formulas based on dynamo wave velocities were used:

\[ G = \rho \times V_s^2, \]  
\[ \mu = \frac{0.5 \gamma V_p^2}{1 - \gamma V_p}, \gamma = \frac{V_s}{V_p} \]  
\[ G = 1.7 \times V_s^2 \times \left(\frac{\sigma_s}{\sigma_{s0}}\right)^{1/3}, \]  
\[ G = 1.3 \times V_{p,s} \times \left(\frac{\sigma_s}{\sigma_{s0}}\right)^{1/6}, \]  

if \( \sigma_s > 0.2 \text{MPa}, V_s \) and \( V_p \), whose zero values are given in Table 1, are determined by the following formula:

\[ V_{p,s} = 1.3 \times V_{p0,s0} \times \left(\frac{\sigma_s}{\sigma_{s0}}\right)^{1/6}, \]  
\[ G = 1.7 \times V_{s0}^2 \times \left(\frac{\sigma_s}{\sigma_{s0}}\right)^{1/3}, \]
Table 1. Reproductive attitudes (average value).

| №  | The name of the ground layer                                                                 | Vₚ₀ [m/s] | Vₛ₀ [m/s] |
|----|---------------------------------------------------------------------------------------------|-----------|-----------|
| 1  | Up to 10-15% with sandstone gravel mixtures (up to 670 m from the base of the dam core)      | 500       | 200       |
| 2  | Bandit and gravel mixture (the core of the dam is 670-726.0 m)                              | 450       | 190       |
| 3  | Gravel sand with a mixture of gravel (transitional prism)                                     | 550       | 300       |
| 4  | Gravel-gravel soil (supporting pillar prism)                                                  | 650       | 400       |
| 5  | Stoneware, represented by large fragments of porphyry l. tuff brewers (dam body)              | 700       | 450       |
| 6  | Tuff boreholes, coarse-grained porphyry, light wind blow, cracked (dam foundation)            | 1450      | 800       |

Figure 2. San Fernando 1971 earthquake accelerogram for 0.43g scale.

The calculation was performed for the normal design face level. It was first introduced by the circular method, using the Morgenstein-Price method, then with the help of the solution of the problem of smooth arc fluctuations.

Figure 3. Scheme for determining the coefficient of stability of the inlet in case of seismic loading, when the water level in the reservoir is projected normal retaining level=726.0m.
Figure 4. Medium hydrostatic voltage distribution at the calculated value of the dam at the maximum value of the foundation acceleration; $p'$

Figure 5. Distribution in the simplified calculation at the maximum value of ground acceleration

In the case of this loading was checked the stability of the dangerous shaft of the dam, which was separated in the previous book. As an example is given the most dangerous whip.

Figure 6. Stability coefficient depending on time

5. Conclusions
Thus, if the stability of the Sarsang dam shafts is not satisfied in the case of the classical circular calculation method, in the contra method it is satisfied. All this means that it is necessary to review the dam safety assessment criteria, not only for the existing dams, but also for the newly designed ground hydraulic structures.

These studies reveal discrepancies between the various methods for evaluating these criteria, but which is a more accurate description of the actual natural processes. It can be noted that in the second case, a larger number of factors are taken into account in the construction of the mathematical model of the natural process than in the first case, which in itself proves the advantage of the second method.

For comparing, in order to satisfy the stability of the Sarsang dam shafts in the first case, it is necessary to increase their slope from 2.08 to 3.50. That is, it will be required to implement approximately 4.0 million high-quality filling of sand-gravel ground on an already built support socket.
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