

# Dynamic Behavior of Earth Dam under Non-Stationary Kinematic Effect

K D Salyamova<sup>1</sup>, X X Tyrdikulov, A A Yangiev, Kh Fayziev



**Abstract**— In connection with the large-scale design, construction and operation of earth dams (the number of which exceeds 50), in the Republic of Uzbekistan which is situated in a seismic region, the mechanics are faced with the task of improving the design methods for calculating main and dynamic loads, including seismic ones. This paper presents a solution to the non-stationary dynamic problem for a particular earth dam taking into account the elastic and inelastic characteristics of soil, the methods used are revealed, instability zones are identified, and the use of the numerical finite element method to solve such problems is justified. The purpose of this work is the development of the methods for solving dynamic problems for earth hydro-technical structures (dams, levees, reservoirs) in a plane statement taking into account the elastic and inelastic properties of soil material. The calculation results make it possible to predict the stress-strain state and determine the vulnerable zones of an earth dam, where loss of stable operation of the structure under dynamic load is possible.

**Keyword:** The purpose of this work is the development of the methods for solving dynamic problems for earth hydro-technical structures (dams, levees, reservoirs)

## I. INTRODUCTION

Design and construction of high rise earth dams require attention to the long-term processes occurring in their body and affecting the quality of construction. There are known catastrophes that occurred during the destruction of the arch dam at Malpas in France or during a landslide in the reservoir of the arch dam Vayont in Italy, the destruction of the earth dam in Teton, USA. In 2017 there was a panic in California (USA) when the threat of the Oroville dam destruction appeared; more than two hundred thousand people were evacuated from the settlements near the dam. This suggests the need to meet the requirements for the safety of dams, including the safety of earth dams.

The current state of the theory of seismic stability of hydrotechnical structures erected from local materials is characterized by the use of refined assumptions and reliable information regarding the choice of computational models that take into account real geometry, design features, piecewise-inhomogeneous soil properties, changing under the influence of filtration flows, the nature of static, hydrostatic and dynamic effects, features of the stress-strain state of the structure.

Identification of the basic laws of stress-strain state and behavior of hydrotechnical structures under seismic effects, taking into account these factors, will allow creating and conducting effective anti-seismic measures that ensure trouble-free operation of the existing and projected earth dams [1,2,3,4,5].

A great contribution to the development of methods for calculating and studying the statics and dynamics of earth dams was made by scientists of the Uzbek school of Mechanics [6,7,8,9,10]. In these papers, on the basis of plane and spatial design schemes, the issues related to seismic resistance of designed and operated structures in areas of Uzbekistan that are subject to high seismic risk are considered. Different soil properties are taken into account - plasticity, water-content, non-linear properties and the construction features of structures.

## II. METHODOLOGY

As is known, the spectral method of calculating structures for seismic effects, regulated by normative documents, is produced on conditional seismic loads, determined using the assumption of elastic deformation of the cantilever model with concentrated mass along the height [11,12]. Such a model justifies itself when calculating high compact in plan structures, the forms of oscillations of which are of a pronounced bending character. Oscillations of massive earth dams are more complex and include not only horizontal shear, but also vertical displacements, which cannot be determined using the cantilever model. Therefore, in the cited papers, calculations of static and dynamic behavior of earth hydrotechnical structures (earth dams) were performed using:

- a plane model (plane-deformable) (Fig. 1 a) representing the cross section of the dam [9,13,14,15];
- a spatial model [7-8], representing the real body of the dam (Fig. 1.b);
- a model consisting of two intersecting planes, one of which represents the transverse section, and the second - the longitudinal section of the dam, which coincides with the cross section of the dam gate (Figure 1 c).

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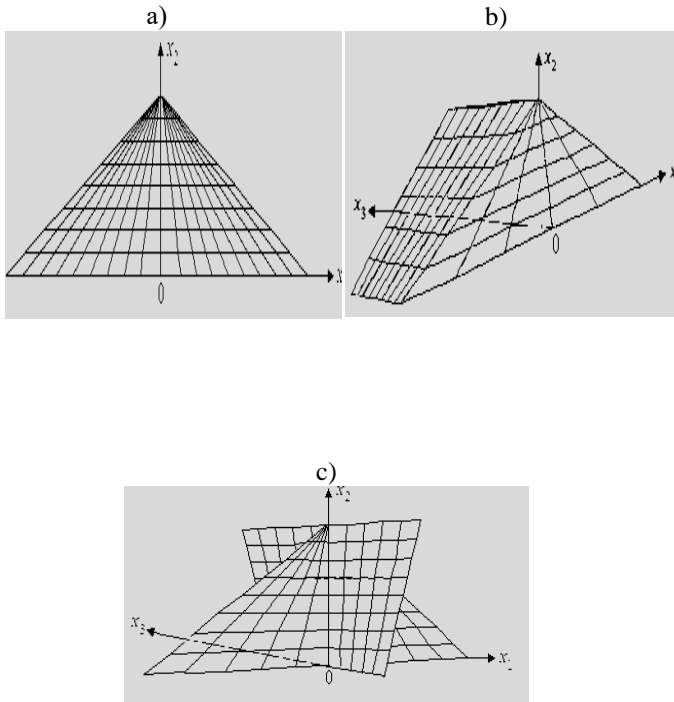


Figure 1. Calculated models of the earth dam on rigid foundation

The lower faces of the calculated models (Figure 1) are fixed, i.e. the dam foundation is assumed to be absolutely rigid. However, to account for non-uniform deformation, subsidence, bulging of part of soil or any other negative manifestation of a weak foundation, the construction models are used with the foundation having sliding side faces (Figure 2), i.e. only the vertical displacement of the infinite foundation strip is taken into account.

Such models were considered in [9], where the effect of a weakened fractured section on the stress-strain state of a high earth dam and the surrounding rocky base with a weakened fractured section was studied; soil parameters were obtained from the results of experimental drilling. The studies had a practical implementation for issuing recommendations for further increase in height of the considered earth dam, taking into account the data of test drilling.

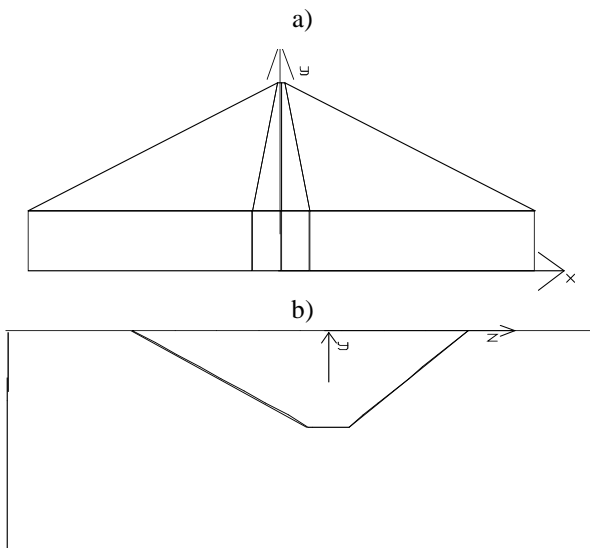


Figure 2 Dam models with foundation: transverse - a; longitudinal (in a canyon) – b

In study of the stress-strain state (SSS) and the dynamic behavior of an earth dam using the models for static, hydrostatic and dynamic effects, the numerical method is used; in most cases, it is the finite element method. This method allows taking into account the real geometry, design features of the objects and various properties of soil under loading and wetting [10,16], such as plasticity, fracturing, geometric and physical nonlinearity. To account for the nonlinear properties of material (soil), special calculation methods were developed based on solving linear and nonlinear systems of algebraic, differential, and high-order integro-differential equations.

The study of the SSS of soil structures (dams) is an extremely difficult task, since the deformation properties of soil depend on many factors: the effective average stress; stress deviation component; applied load; water-content; degrees of fracturing, etc. Nonlinear laws of soil deformation in a structure, implemented in the specified calculation programs include consideration of structural damage at volume deformation; water-content; geometrical (for high dams) and physical nonlinearity under loading, as well as dry and viscous friction.

The developed methods of static and dynamic calculation and the solution of complex problems of the SSS and dynamic behavior of earth structures allowed to analyze the effect of hydrostatic pressure, viscosity, nonlinear strain and water-content in soil on the SSS of specific earth dams of Uzbekistan [13,14,15,16,17,18]. The analysis of possible destruction zones in dams with a variety of static and dynamic effects allows a reasonable and economical approach to the issues of operation of selected earth protective structures - this determines the relevance and practical value of the carried out studies.

To create a mathematical model the following parameters are used [15,16,17,18,19]:

1) The variational principle of the minimum total energy of the system:

$$\delta\Pi - \delta'W = 0 \tag{1}$$

where  $\delta\Pi$  is the increment of potential energy of the system, and  $\delta'W$  is the sum of the work of external forces on possible displacements;

2) The equation of state expressing the relationship between the components of stresses  $\sigma_{ij}$  and strains  $\epsilon_{ij}$  for an elastic medium

$$\sigma_{ij} = \lambda \epsilon_{kk} \delta_{ij} + 2\mu \epsilon_{ij} \tag{2}$$

3) The Cauchy relations, connecting strains with displacements

$$\epsilon_{ij} = \frac{1}{2} \left( \frac{\partial u_i}{\partial x_j} + \frac{\partial u_j}{\partial x_i} \right) \tag{3}$$

4) Boundary conditions on a rigid foundation, showing the absence of possible displacements  $\delta \bar{u}$  :

$$\delta \bar{u} = 0 \tag{4}$$

5) An external effect is represented by volume forces - P (by weight) applied throughout the entire volume of the structure, and surface forces  $\bar{f}$ , acting on the parts of the structure surface, being a hydrostatic pressure.

Here  $\bar{u} = \{u_i, u_j\}$  are horizont

al and vertical displacements of a body point with coordinates  $\{x_i, x_j\}$ ;  $\sigma_{ij}, \epsilon_{ij}$  are the components of the stress and strain tensor;  $\lambda, \mu$  are the Lamé constants.

For the numerical solution of the problem, the finite element method [15,16,17,18,19] is used, which is currently one of the main numerical methods in the mechanics of a deformable rigid body. This method is invariant with respect to the geometry of the object under study and mechanical characteristics of material (earth and concrete). In addition, the finite element method is characterized by the simplicity of accounting the structure interaction with environment, various mechanical (static and dynamic) loads, and various types of boundary conditions.

Mathematically, the problem of unsteady forced oscillations is reduced to solving a matrix system of inhomogeneous second-order differential equations with the right-hand side being a function of time

$$[M]\{\ddot{q}\} + [C]\{\dot{q}\} + [K]\{q\} = \{P(t)\} \tag{5}$$

where [M] is the mass matrix; [K] is the stiffness matrix;  $\{P(t)\} = [M]\{\ddot{u}_0\}$ ; the damping matrix [C] describes the internal friction caused by the viscosity of medium. The question of choosing the type of matrix [C] is discussed below.

The solution of the system of equations (5) under appropriate initial conditions can be obtained by the Newmark method [20]. The Newmark method is based on

expansions of  $q(t_i + \tau)$  and  $\dot{q}(t_i + \tau)$  into the series in powers of  $\tau$  (the integration step):

$$q(t_i + \tau) = q_i + \tau \dot{q}_i + \frac{\tau^2}{2} \ddot{q}_i + \alpha \tau^3 \ddot{q}_i \dots$$

$$\dot{q}(t_i + \tau) = \dot{q}_i + \tau \ddot{q}_i + \beta \tau^2 \ddot{q}_i \dots$$

where  $\alpha$  and  $\beta$  are chosen so that unconditional convergence of the integration process is ensured:  $\beta \geq 0,5$ ;  $\alpha \geq 0,25(\beta + 0,5)^2$ .

The solution for the nodal displacements  $q_{i+1}$  at the end of the i-th time step is determined from an algebraic system of equations [20]:

$$[A]\{q_{i+1}\} = \{P_{i+1}\},$$

solved by the Gauss method.  
Here

$$[A] = [K] + [C]\beta/(\alpha\tau) + [M]/(\alpha\tau^2);$$

$$\{P_{i+1}\} = \{R_{i+1}\} + [M] \left[ \frac{\{q_i\}}{\alpha\tau^2} + \frac{\{\dot{q}_i\}}{\alpha\tau} + \left( \frac{1}{2\alpha} - 1 \right) \{\ddot{q}_i\} \right] + [C] \left[ \frac{\beta\{q_i\}}{\alpha\tau} + \left( \frac{\beta}{\alpha} - 1 \right) \{\dot{q}_i\} + \frac{\tau}{2} \left( \frac{\beta}{\alpha} - 2 \right) \{\ddot{q}_i\} \right],$$

$\{q_i\}, \{\dot{q}_i\}, \{\ddot{q}_i\}$  - are the displacements, velocities and accelerations of nodal points found at the previous time step.

The described algorithm of the Newmark method is applied to solving problems on unsteady forced oscillations of the considered earth dams, the dynamic effect for each structure is chosen individually.

The calculations were made on the example of the Rezaksay reservoir located in the Rezaksay valley (a tributary of the Syr Darya river) in the Chust district of the Namangan region of the Republic of Uzbekistan. The dam, which generates a reservoir, is made of a blind earth fill with an inclined core of sandy-loamy soil and retaining prisms of gravel-pebble soil (Fig. 3). The maximum height of the dam in the channel part is 80 m, the length of the crest is 3323 m, the greatest rise in the level of the reservoir is 77 m.

Retaining prisms of the dam are filled by gravel-pebble soil with sand filling and rolling to the density of dry soil  $\rho = 2.1 \text{ t/m}^3$ . The rate of the dam slopes is determined by calculations and is  $m_1 = 2.5$  for the upper slope, and  $m_2 = 1.9$  for the lower slope.

The inclined core is made of a mixture of loam and sandy loam with rolling to the density of dry soil  $\rho = 1.7 \text{ t/m}^3$ . The base of the core along the entire length of the dam is an array of aleurolite and sandstone, covered with layers of soil of various thickness, consisting of pebbles, loams and sandy loam.

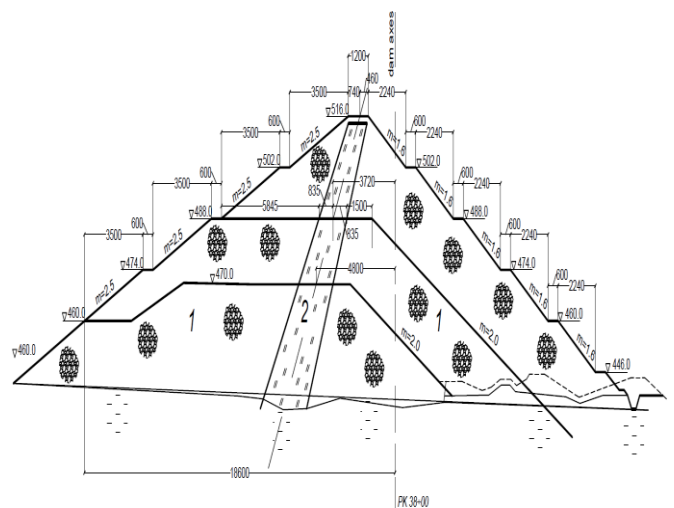


Figure 3. Rezaksay earth dam

- 1- gravel-pebble soil with sand filling; 2 - a mixture of loam and sandy loam.

III. RESULTS

Consider non-stationary forced oscillations on the example of the above earth dam. The initial conditions are assumed to be homogeneous (zero):

$$\text{at } t=0: \{q_0\}=0, \{\dot{q}_0\}=0.$$

Let the kinematic effect be represented as a harmonic function with eigenfrequency of the structure, and its duration is 2 seconds. The entire time interval is 3-4 seconds:

$$\ddot{u}_0 = \begin{cases} A \sin(2\pi p t) & 0 \leq t \leq 2c \\ 0 & t > 2c \end{cases} \quad (6)$$

After the effect cessation a free oscillation mode is set in the structure. In 2 seconds, the construction in the resonant mode makes 6-7 oscillations, by analyzing which it is possible to draw the necessary conclusions about its dynamic behavior in seismic process.

This kinematic effect with a frequency equal to the basic frequency of natural oscillations of the structure ( $p=\omega_1=3,1 \text{ Hz}$ ) is used to demonstrate the dynamic behavior of the structure in a dangerous resonant mode. Besides, it should be noted that such a harmonic effect with a period of  $T = 0.05 \div 0.3 \text{ s}$  can be classified as a seismic one, since its frequency range coincides with the frequency range of seismic effects. For example, the predominant period of the 1976 Gazli earthquake was about  $T = 0.1 \text{ sec}$ . Thus, an artificially chosen effect may be a substitute for a real accelerogram.

In the absence of energy dissipation ( $[C] = 0$ ), system (2) describes the motion of an elastic dam without attenuation. The horizontal and vertical displacements of a point near the dam crest under its own weight and given kinematic action, obtained in this case by the Newmark method with zero initial conditions are shown in Figs. 2.a. The frequency of the kinematic (harmonic) effect at the base of the dam is equal to the natural oscillation frequency of the Rezaksay dam  $\omega_0= 3.1 \text{ sec}^{-1}$ , and the amplitude of the effect  $A=0.1$  corresponds to the soil acceleration at a seven-point earthquake.

As expected, under the effect of a frequency equal to the basic natural frequency of the structure, i.e.  $p=\omega_1$ , the latter oscillates with a linearly increasing amplitude, which by the end of the effect reaches 0.1 m (10 cm) for horizontal vibrations and 0.02 m (2 cm) for vertical ones. Such a linear increase in the amplitude when the frequency of the effect coincides with the basic oscillation frequency of the structure (resonance) is predictable and proves the reliability of the results obtained according to the developed program.

The same figure shows the distribution of maximum displacements (horizontal - fig.4.b and vertical - fig.4.c) over the entire section of the dam for the entire period of the effect.

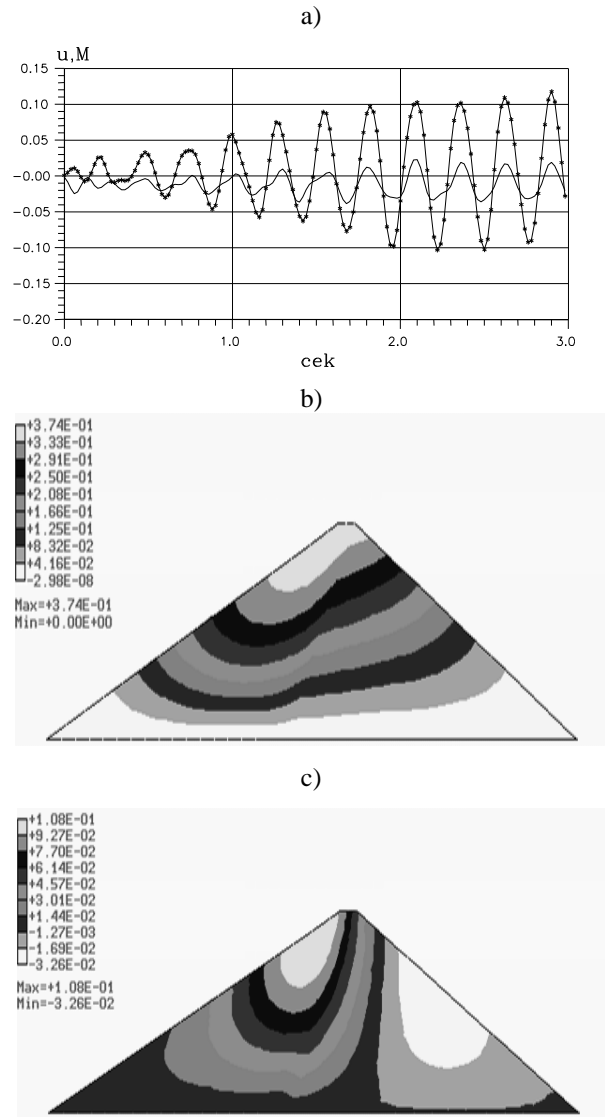


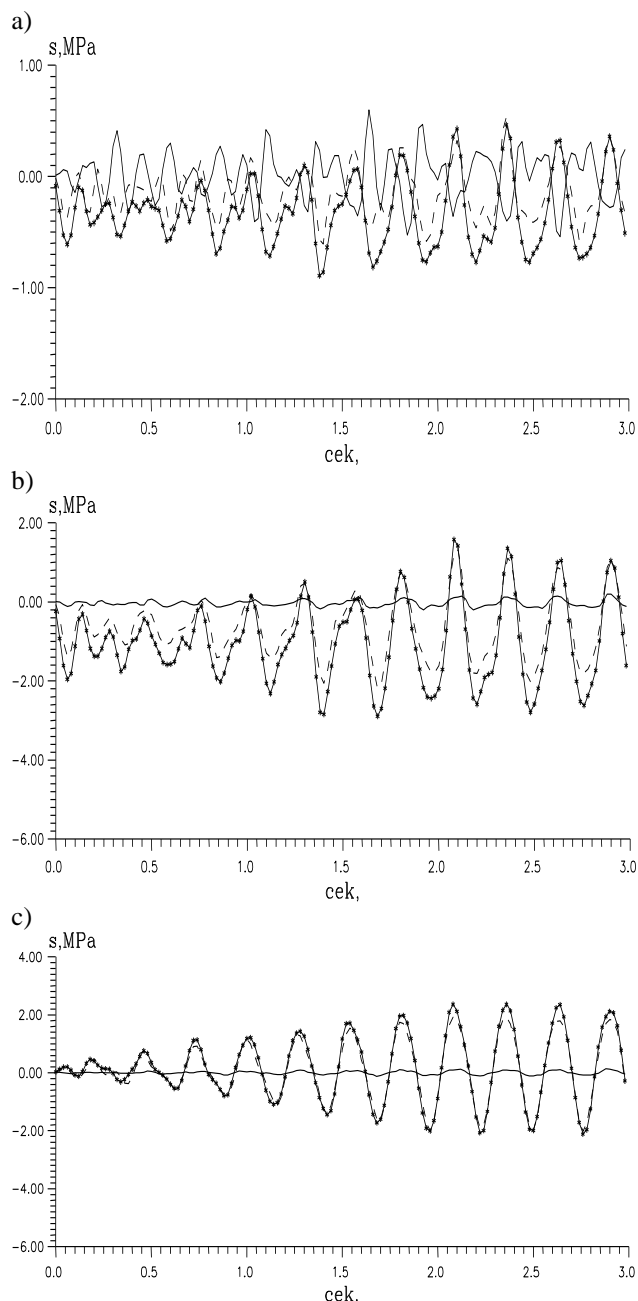
Figure 4. Horizontal (—\*—) and vertical (—) displacements of the center point (a) and the distribution of maximum horizontal (b) and vertical (c) displacements of the dam section for the entire period of the effect

After the effect cessation, the free oscillation mode is set in the dam; vertical oscillations occur relative to the position of static equilibrium, determined by the weight of the structure (Fig. 4 - thin line).

Horizontal displacements at the first resonance, as shown by the results of Figs. 4 b, c, reach the maximum value on the dam crest and uniformly decrease to the rigid foundation. The displacements of the upper prism are positive, and the lower ones are negative, i.e. during the shift, the slope and the near-crest zone of the upper prism shift upwards, and the symmetrical part of the lower prism moves downward.

The results in Fig.4. obtained without taking into account dissipation in soil, therefore, free oscillations of the dam occur with a constant amplitude.

Graphs of changes in the normal - horizontal and vertical one, and tangential stresses for the entire period considered are shown in Fig.5.



**Figure 5. Change of the normal - horizontal (a) and vertical (b), and tangential (c) stresses at the points of central section (—\*—\*— at the bottom; - - - in the middle; ——— at the top) under resonant oscillation mode**

#### IV. DISCUSSION

Graphs of stress changes over time show an increase of vertical ( $\sigma_y$ ) and tangential ( $\tau_{xy}$ ) stresses in this mode during the entire time of load effect. The maximum stress values (taken as an absolute value) are observed at points near the foundation. The predominance of negative values of vertical stresses means that oscillations of earth structure occur relative to the level of static equilibrium, characterized by structure compression under its own weight. Over time, the amplitudes of vertical stresses at the bottom and in the center (the —\*—\*— and - - - lines in Figure 5b) extend into the positive half-plane, which means that positive tensile vertical stresses appear, weakening soil connection with the foundation.

Next, consider the dynamic behavior of earth dam, taking into account the dissipative properties of soil under the same dynamic effects.

The reasons that lead to energy dissipation can be caused by energy loss to the environment (“external” friction), and the loss caused by internal processes in material of the system (“internal” friction). In the first case, it is believed that dissipative forces are proportional to inertial forces, the second case is related to the viscous behavior of material under strain.

To describe the absorbing, dissipative properties of soil, and to obtain a resolving system of equations, the Kelvin-Voigt dynamic model of viscoelastic medium is applied.

$$\sigma_{ij} = \lambda\theta\delta_{ij} + 2G\varepsilon_{ij} + \lambda'\dot{\theta}\delta_{ij} + 2G'\dot{\varepsilon}_{ij} \quad (7)$$

where  $\sigma_{ij}$  is the component of stress tensor;

$\varepsilon_{ij}$ ,  $\dot{\varepsilon}_{ij}$  - are the components of strain tensor and strain rate tensor;

$\lambda, G$  - are the Lamé constants;

$\lambda', G'$  - are the corresponding viscosity coefficients of medium;

$$\theta = \frac{1}{3}(\varepsilon_1 + \varepsilon_2 + \varepsilon_3) \quad \dot{\theta} = \frac{1}{3}(\dot{\varepsilon}_1 + \dot{\varepsilon}_2 + \dot{\varepsilon}_3) \quad \text{is an average strain and strain rate;}$$

$$\delta_{ij} = \begin{cases} 1, & i = j \\ 0, & i \neq j \end{cases} \quad \text{- is the Kronecker symbol.}$$

The use of such a model in calculations of structures made of earth materials on seismic effects makes it possible to take into account the energy absorption in soil due to the viscosity of material, the friction between solid particles, the water-soil skeleton interaction under irreversible plastic strain, etc. In addition, this model makes it possible to evaluate absorptive capacity of earth structures depending on the frequency spectrum of the structure.

The description of viscoelastic behavior is achieved by the representation of the components of strain and the average deformations in a complex form  $\varepsilon_{ij} = \varepsilon_{0ij} \exp(i\omega t)$ ,  $\theta = \theta_0 \exp(i\omega t)$ , as a result, the strain rates and the rate of volume change are equal to

$$\dot{\varepsilon}_{ij} = i\omega\varepsilon_{0ij} \exp(i\omega t), \quad \dot{\theta} = i\omega\theta_0 \exp(i\omega t).$$

To obtain an explicit expression for the dissipation matrix [C], which enters the equation (5), we transform (7) using the complex modules

$$\lambda(i\omega) = \lambda - i\omega\lambda'; \quad G(i\omega) = G - i\omega G'$$

Then equation (26) with complex modules takes the form similar to Hooke's law

$$\sigma_{ij} = \lambda(i\omega)\theta\delta_{ij} + 2G(i\omega)\varepsilon_{ij}$$

,an explicit expression for the dissipation matrix [C], which enters the equation (5) before the derivatives of nodal displacements, is obtained as

$$[C]=\eta[K]$$

where  $\eta = \lambda' + 2G'$  - is the positive constant.

Thus, the use of this model in the finite element discretization of a structure leads to a resolving system of differential equations

$$[M]\{\ddot{q}\} + \eta[K]\{\dot{q}\} + [K]\{q\} = \{P(t)\} \quad (8)$$

where  $\eta$  - is the viscosity coefficient.

In (8), the matrix of damping coefficients is proportional to the matrix of quasi-elastic coefficients; this case is called internal friction and is related to the manifestation of viscous properties in material. If to transform (8), multiplying it from the left by the matrix inverse to the mass matrix ([M] - 1), we get

$$\{\ddot{q}\} + \eta[M]^{-1}[K]\{\dot{q}\} + [M]^{-1}[K]\{q\} = [M]^{-1}\{P(t)\}$$

or, given that  $[M]^{-1}[K] = \text{diag}(\omega_i^2)$  - is the diagonal matrix of squares of natural frequencies the following system of separate equations is obtained

$$\{\ddot{q}\} + \eta \text{diag}(\omega_i^2)\{\dot{q}\} + \text{diag}(\omega_i^2)\{q\} = [M]^{-1}\{P(t)\} \quad (9)$$

To select the value of  $\eta$  we use known data, according to which the values of  $0,2 \leq \psi \leq 0,35$  are given for the coefficient of the internal absorption of soil  $\psi$ . Then, from the formula relating the coefficient  $\psi$  to the coefficients of friction (coefficients of the derivative of displacements  $\eta$ ) and frequencies ( $\omega$ ), we obtain

$$\psi = \frac{2\pi\eta\omega_i^2}{\omega_i}, \text{ hence } \eta_i = \frac{\psi}{2\pi\omega_i}$$

Taking into account the range of variation of  $\psi (0,2 \leq \psi \leq 0,35)$  and the spectrum of the principal frequencies (3÷5.4 Hz), the following limits of variation of the coefficient  $\eta$  for the soils of Rezaksay dam are obtained

$$0,006 \leq \eta \leq 0,0175,$$

from which we choose the mean value  $\eta = 0.01$ , used later to calculate the dynamic behavior of soil structure (Rezaksay dam) on dynamic load. The effect, as before, is harmonic with a frequency that coincides with the main oscillation frequency of the structure. Initial conditions are zero. The method for solving system (8) is the Newmark method [20]. The result of the solution is the displacements of the nodal points of the structure shown in Fig.6.

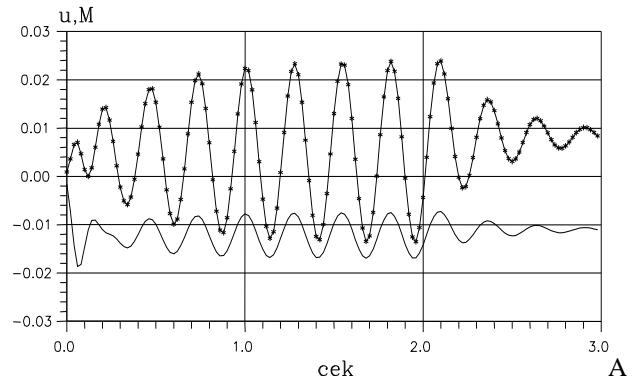
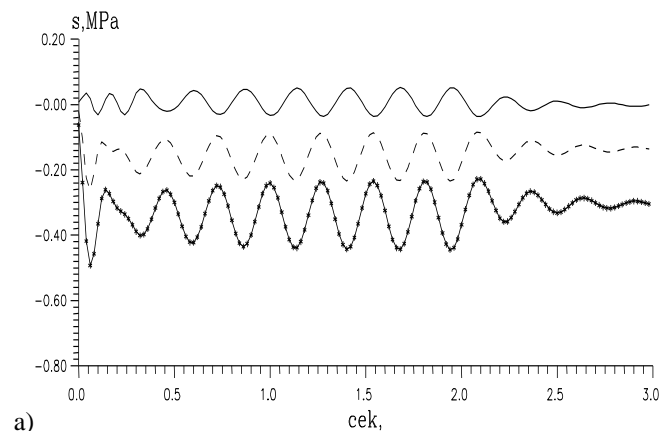


Figure 6. Horizontal (—\*—) and vertical (—) displacements of the Rezaksay dam crest, taking into account internal friction

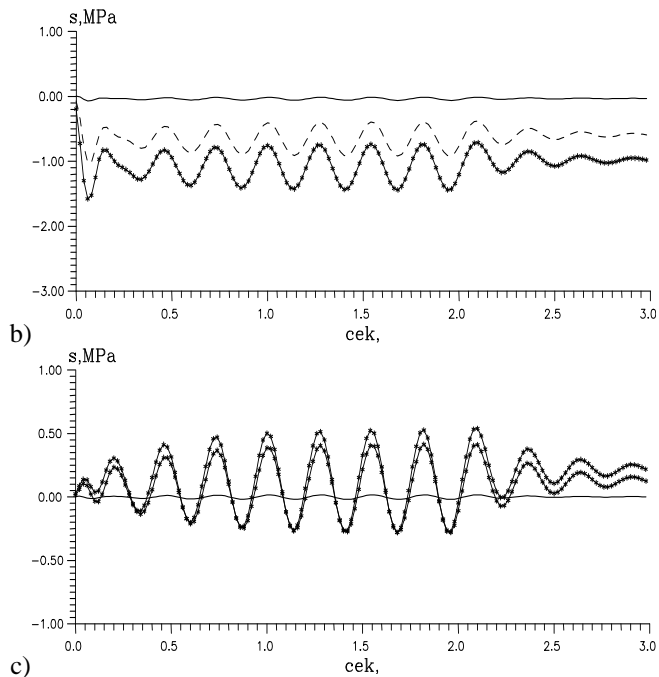
A comparative analysis of the behavior of an elastic dam (Fig. 5) and a dam with internal friction in material (Fig. 6) shows that in the second case, both horizontal and vertical oscillations occur with an amplitude much lower than the amplitude of the elastic case. Moreover, even under kinematic action, the amplitude of oscillations of a dam with internal friction does not constantly increase. This indicates a change in frequency spectrum of a structure with damping properties of soil. In other words, the absence in this case of a resonance, i.e. unlimited rise in the amplitude of oscillations, indicates a change (decrease) in the basic frequency of the system, caused by rheological processes in the soil of the dam.

Horizontal oscillations occur relative to the neutral position (line —\*— in Fig. 6), and vertical oscillations in relation to the position of static equilibrium, determined by the displacement of the structure under its own weight (line — in Fig. 6). After the effect cessation ( $t > 2$  sec), the oscillations quickly damp out, and the strained state of the dam at the end of the process is characterized by an axis shift in horizontal direction (horizontal displacements of the crest are set above the neutral line) and vertical settlement at the level of static equilibrium.

The change in the components of the stress state under kinematic effect with the principal frequency of the oscillations of elastic structure, taking into account internal friction in soil, is shown in Fig.5.



a)



**Figure 7. Variation of horizontal (a), vertical (b) and tangential (c) stresses in central section (—\*— at the bottom; - - - in the middle; — at the top) of the Rezsaksay dam with internal friction in soil under harmonic effect with own frequency**

Therefore, an account of internal friction in soil reduces the amplitude of oscillations and the level of stresses in earth structures. In addition, the effect of weakening of structure-foundation connection, noted in the elastic case, disappears, as indicated by the output of the amplitude of vertical displacements into the positive half-plane. Here (Figure 7 b) such a phenomenon is not observed.

## V. CONCLUSIONS

The statement of the problem of forced oscillations of an earth dam under dynamic effect with and without taking into account internal friction in soils is given; an algorithm for solving non-stationary problems by the numerical finite element method using the Newmark method is presented. The analysis of numerical results is given. The obtained dependences of the displacements and stresses of the vulnerable points of earth structure over the time of dynamic effect showed that taking into account internal friction in soil leads to a decrease in the amplitude of the forced vibrations, which is necessary in calculations.

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## Dynamic Behavior of Earth Dam under Non-Stationary Kinematic Effect



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